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WATERWAYS AND HARBORS DIVISION
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TELEMETERING HYDROLOGIC DATA

Francis P. Hanes*
(Proc. Paper 1365)

SYNOPSIS

Automatic equipment is described to fulfill a requirement for optimized operation of reservoirs and other flood control structures by allowing rapid and unattended reporting of river levels and precipitation amounts from remote points. The emphasis is on design of telemetering devices and system engineering for VHF radio transmission of the data.

INTRODUCTION

A rapid and accurate means of gathering basic data is a vital prerequisite to the effective operation of flood control installations. Regulation of reservoirs for maximum benefit depends upon a knowledge of changes in hydrologic conditions at the time of their occurrence. The fundamental quantities involved are precipitation amounts and stages of streams and rivers. Realizing the need for development of equipment to measure and transmit the necessary information, the Office, Chief of Engineers, authorized a program "Development of Hydrologic Equipment" in 1951. A program of this nature was indicated because commercial equipment manufacturers were reluctant to risk engineering and development costs in view of the somewhat limited market potential. The development program was assigned to the U. S. Army Engineer District, Louisville, Ohio River Division.

General Considerations

The District Offices of the Corps of Engineers have the primary responsibility for construction and operation of flood control structures. The operation

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is integrated by means of radio voice communication networks so that the District Office may issue instructions to, and receive reports from the several damtenders within the District. Since hydrologic data required can be logically divided into the various drainage basins involved, and since communication facilities already exist, it is practicable to use the dam operating room in each case as the collection center for the hydrologic network. In this manner the damtender collects the raw hydrologic information and transmits it to the District Office where it is used to formulate an over-all operation plan. Based on this, releases from a particular reservoir are scheduled.

A system of components designed for telemetering hydrologic data consists of keyers, a transmission network, and a means of indicating or recording the magnitude of the remotely measured variables. Normally, keyers and recorders are considered together since they must be compatible, and usually one recorder is used to serve an entire telemetering network in a particular reservoir drainage basin.

Basically, a keyer is a device used to convert the measurement of some variable into a form suitable for transmission and subsequent recording or indication by visual or audible means. Telemetering a magnitude of electrical current or voltage analogous to a measurement places stringent requirements on the transmission link. Therefore, an alternate method of keying, or converting the measurement into a digital pulse form, is generally preferred. The keyer, then, is required to generate a series of pulses or a sequence of pulses representing the water stage or precipitation amount involved.

While pulse-count and pulse-code systems are basically digital, an analog pulse system is usable although not compatible with a digital recorder. Digital systems are logically divided into two groups: (1) Straight Digital Systems wherein a group of pulses is generated to represent each digit of a number as shown in Figure 1. This system is desirable when 3 or 4 digits, such as river stage to tenths or hundredths of a foot, are required; (2) Accumulative Digital Systems wherein the number to be transmitted is represented by that many pulses, as shown in Figure 2. This system lends itself to transmission of rainfall amounts where there is a total range of, say, 30 inches with least count of 0.1 inch, giving a total pulse output not exceeding 300 pulses. Digital print recorders are available for both systems. Keying speed is of some concern particularly in the accumulative system. Available digital print recorders suitable for this service have response rates up to about 10 pulses per second. Maximum rates of about 5 pulses per second are usually selected since this rate will permit gathering information from an entire network consisting of several stations in a reasonable length of time and is well within the response characteristics of tone equipment used in the transmission link. One specialized case of a digital water stage telemetering device is the split-digital system. In this method pulses are generated in two separate accumulative groups (Figure 3) to give a stage reading of 15.13 feet. A digital print recorder may be adapted to operate with this device.

It should be noted here that in using digital systems in the manner described, the concept of accuracy based on percentage of full scale hardly applies. Accuracy depends to some extent on the system selected. For example, in the straight digital system shown in Figure 1, missing a pulse affects accuracy to a very large extent if the pulse is lost in the tens column or units and to a lesser extent as the pulse train nears completion. On the other hand, with the accumulative system the loss of a pulse or so only

represents that much error in terms of least count of the device. For this reason accumulative digital systems are used where practicable.

When there are several keyers in a network being served by one recorder, it is necessary to have positive identification of the various stations. This may be done by having each keyer generate a discrete number of pulses (different for each keyer), which pulses may be channeled into a lettered identification wheel in the digital print recorder.

Since there are a limited number of radio channels available for this service, it is necessary to stack as many channels of telemetering on one radio channel as practicable. This is accomplished by use of selective tone equipment to transfer the pulse information from a number of keyers all operating on a single radio channel. Tone frequencies are selected to fall within the audio pass band of the transmission equipment. Best operation is had by using frequency-shift tone equipment wherein a center frequency tone is shifted, say, 30 cycles above and below the center frequency to convey the intelligence. Suitable modification of commercially available tone equipment of this type results in a tone channel where two discrete bits of information may be had with a single channel. This modification, known as "center-neutral keying," is useful in supplying the necessary information to the recording device to have it separate the station identification pulses from the information pulses, and print out the stored information on completion of the information pulses.

If we consider the audio pass band (Figure 4) of commercially available frequency modulated radio equipment suitable for this service, it is seen from a plot of response versus audio frequency that the usable frequency spectrum lies essentially between 300 and 3000 cycles per second. Figure 5 shows a method of utilizing this audio band for telemetering functions together with a relatively narrow voice communication channel. Voice communication is provided only as an aid in installation and maintenance since the primary function of the network is telemetering. Filters are used to prevent false operation of telemetering equipment by voice operation or noise.

Often it is desirable, or even necessary, to provide an emergency power plant at each station, or at selected key stations, to take over in event of commercial power failure. It then becomes desirable to provide a remote indication of running time of the power plant which, again, is a telemetering function and requires a tone channel. In this connection, bottled gas is the preferred fuel for this service to relieve problems of cold-weather starting and fuel storage. The tone channels marked "supervisory circuits" may be allocated to operation of remote running-time equipment.

The radio network for hydrologic-data telemetering usually represents a design compromise based principally on the locations from which data are desired. Precipitation stations are generally located in terrain favorable for radio transmission, while river stage locations make line-of-site radio paths difficult to obtain. A radio survey is usually made to determine necessity for, and location of, repeater stations to result in a transmission link with sufficient signal strength to be unaffected by seasonal variations in propagation characteristics and other forms of interference.

Radio equipment for this service is readily available commercially and no particular problems are involved with the exception of repeater station design. With an interrogation type system using more than one repeater station, special precautions must be taken to prevent "repeater lock-up." This will occur

when each repeater is energized in such manner that there is an energy loop. This situation is prevented by use of selective directional antennas with coaxial switching. The other problem common to repeaters is audio deterioration due to receiver desensitization resulting from operation of a receiver and transmitter at the same time. Antenna separation and frequency separation together with use of cavities and shields will result in satisfactory repeater station operation.

When a network is installed on an interrogation or on-call basis, it is convenient to include an automatic programming device to sequentially interrogate the stations at predetermined time intervals with a master time clock at the data collection location. Individual station manual interrogation may also be provided.

An experimental network was in operation in the Bluestone Dam drainage basin which proved its value during hurricane Hazel in 1954. Using the precipitation information obtained in the network the Huntington District Office was able to predict flood crests along the Greenbrier River, an uncontrolled river below the dam, to an almost unbelievable accuracy of 0.02 foot.

Keyers

Many types of keying devices for precipitation and river stage have been evaluated. Details of operation and relative merits of these units have been reported in the form of project bulletins issued by the Ohio River Division, Corps of Engineers. Two keyers are discussed in this paper, one for river stages and one for precipitation, representing the best available types at present. Both are compatible with digital print recorders although, as will be shown, the digital systems employed are different, requiring at present one recorder to serve the river stage units and another somewhat similar recorder to serve the precipitation units. This is due to the fact that the river stage units use the split digital system while the precipitation devices utilize the accumulative digital system of pulse generation.

River Stage Keyers

These units are commercially available float-operated keyers normally manufactured for tank-farm-level telemetering in the petroleum industry. The standard keyer is supplied with explosionproof housing and fittings and gives indication of level in feet, inches, and eighths. A minor modification may be made to convert the keyer to the decimal system with least count of one-hundredth foot instead of one-eighth inch. Input torque requirements are low such that the device may be operated with a six-inch-diameter float. The device is intended for use with a two-dial indicator in lieu of a recorder with provision for a third indicator for station identification. The indicator unit will handle up to twenty remote keyers. It is built for and usually used as a wire line telemetering system. The line requirements are not stringent and "teletype" grade of service is usually satisfactory. Figure 6 shows the keyer with explosionproof bell housing removed. Figure 7 shows the indicator (this unit was modified for decimal indication). Minor electrical changes make the keyer suitable for radio telemetering using selective tone equipment. It is possible, of course, to use the indicator unit in lieu of, or together with, a digital print recorder.

In operation, the keyer converts shaft rotation to pulse output in a split digital manner. A digital print recorder, Figure 8, for this application would consist of two sets of print wheels connected with a star-wheel mechanism similar to mechanical stroke counters. The print wheels have raised numbers so that when struck by a hammer with typewritter ribbon and paper passing between, the numbers are printed on the paper. The only special requirement over the standard print recorder is that the four digit wheels be set up in accumulating pairs, being advanced by solenoids with electrical contacts as in the standard unit. Station identification is handled with a lettered wheel. Date and time print at the time of recording is a standard feature available on print recorders. The separation of shift and count functions for operation of the recorder is handled on a time-pulse rate basis in the indicator furnished. This will be described in more detail later. Briefly, however, the recorder requires two types of electrical contact information, namely, shift pulses and count pulses. The shift pulses transfer the count, or information, to the proper solenoid for accumulating information on the lettered or numbered wheels. Finally, a shift pulse is used to cause the recorder to print out the stored information and reset all wheels so as to be ready for another complete cycle, as above.

The keying device is manufactured by Shand and Jurs, Berkeley, California, and has a range of 0 to 60 feet with least count of 0.01 foot. The digital print recorder is manufactured by the Streeter-Amet Company of Chicago, Illinois.

Precipitation Keyers

A device for telemetering precipitation has been developed in the Corps of Corps of Engineers program which is considered superior to any yet tested. The advantages include: rugged construction; simplicity of operation; large capacity; low cost; absence of springs and balance linkages and resulting inherent torque difficulties; direct reading; ease of winterizing; and use for frozen precipitation.

The device consists of a collector ring and receiver to catch and store precipitation, and the necessary electrical, mechanical, and electronic components to measure and code for transmission, by radio or wire line, the collected precipitation. The measurement is obtained by a motor-driven probe which travels from a reference point to the water surface, generating (in present models) during its travel a pulse for each 0.1 inch of precipitation. On contact with the water surface the probe is returned to its starting position. Recording of the transmitted pulses is readily obtained by commercially available counter-printers.

Proper operation of the counter-printer depends upon two types of pulse information — step pulses and count pulses. Step pulses (called "B") channel the count pulses (called "A") into the proper counter circuit. In the example (Figure 9) pulse B-1 (step pulse) sets up the counter to receive a predetermined number of count pulses (A-1) stepping one print wheel in the counter, thus identifying the station. The second step pulse, B-2, sets up the counter to receive and accumulate a number of count pulses (A-2), each pulse representing 0.1 inch (or some other unit) of precipitation. The third step pulse, B-3, causes the counter to print the stored information (station identification and precipitation), the time and date, and resets the counter for the next cycle. Figure 10 shows a sample record obtained in this manner.

The telemetering device consists of the following basic components: collector ring, receiver, electric and electronic panels, and cover. The model shown in Figure 11 is the 15-inch capacity gage now in service in the West Fork of Mill Creek Reservoir Network, near Cincinnati, Ohio. Figure 11 shows the model with the electronic subassemblies removed. Components labeled are station identification unit motor (M-1), reference switch (SW-1), pulse generating switch (SW-2), and transfer switch (SW-3). The main drive motor (M-2) is connected by gears to two lead screws, one of which is inside the receiver and the other outside. Also connected by gears to the main drive motor is the pulse generating switch (SW-5). The follower nut on the outside lead screw operates the upper and lower limit switches (SW-4 and SW-5). The follower nut on the inside lead screw carries the insulated probe and retractable cord.

The operating cycle of the keyer is begun with an interrogation pulse originating at the collection center. By means of the relay and electronic circuits, the device generates a station identification pulse train. After this, the main drive motor is energized to drive the probe downward. When the follower nut passes the upper limit switch the mechanically synchronized pulses, generated by SW-5, are gated out and key the audio tone to modulate the radio transmitter. These pulses continue, representing discrete increments of travel of the probe, until contact is made with the water surface in the collector can. The drive motor is then reversed so as to withdraw the probe. The pulse gate is closed upon contact with the water. The amount of precipitation in the collector can is the difference between the pulses thus generated and the capacity of the can.

For winter operation, thermostatically controlled electric heaters are mounted under the collector ring and around the receiver to melt frozen precipitation and maintain it in liquid state.

Radio Equipment

Commercial radio equipment is entirely satisfactory for this service. Antennas, towers, and fittings are likewise available. In the Government 169-172 mc hydrologic band, maximum transmitter power allowable is 50 watts. Frequency modulation is invariably employed because of superior performance and relative insensitivity to atmospheric disturbances. Total carrier deviation with modulation is 36 kilocycles. Transmitter specifications usually include provision for positive deviation limiting at 15 kilocycles. Transmitted frequency is controlled with a crystal followed by several RF multiplier stages. One commercial version uses a total multiplication of the basic crystal frequency of 24 times.

Receivers are available with sensitivities of 0.6 microvolt with narrow band operation controlled by crystals. The high sensitivity and selectivity are obtained with superhetrodyne double-conversion techniques.

Figure 12 is a block diagram of a typical transmitter showing oscillator, modulator, doubler, tripler, doubler, combined double-driver, and power amplifier stage.

Figure 13 is a block diagram of a typical receiver showing RF amplifiers, first converter with crystal-controlled oscillator stage, high intermediate-frequency amplifiers, second crystal-controlled converter, low intermediate-frequency amplifiers, limiter stages, discriminator, squelch, and audio

circuits. Manufacturers of this equipment include Motorola, General Electric, and RCA.

Tone Equipment

As previously mentioned best operation is had with frequency-shift tone equipment for the same fundamental reason that FM radio is superior. Well designed, amplitude-type tone equipment has been used successfully although it is more sensitive to voice and random noise falsing. Delay circuits may be employed with the amplitude sensitive type to make this equipment satisfactory for interrogation purposes such that the tone must be present for some specified period of time thus making the circuit insensitive to voice operation and random bursts of noise. Motorola and General Electric manufacture frequency-shift tone equipment while the Hammarlund Manufacturing Company makes well designed, amplitude-sensitive tone equipment.

Bottled-Gas Operated Telemetering Station

To determine the reliability and operation characteristics of bottled-gas power generators, as compared to gasoline generators, a telemetering station, independent of commercial power, has been installed in the Louisville District in the Rough River Reservoir Network. The station contains the following equipment (see Figure 14).

a. Water stage keyer.

b. Nickel-cadmium battery, 12 volt, 80 amp-hr.

c. Mobile 2-way FM radio equip. - 50 watt.

d. Generator-bottled gas, 12 volt, 500 watt.

e. DC time clock.

f. Ampere-hour meter, with contacts.

g. Control panel, with metering facilities for voltage and current.

h. Tone equipment.

i. Inverter and vibrator power supply.

Operation of the station is controlled by the time clock in such a manner that for selected periods during the day the clock circuit closures make the station available for interrogation. There is also provision for a float-operated switch to supercontrol the time clock and make the station available for interrogation at all times when the river stage is above a predetermined criterion. Control circuitry provides for operation of the gas-driven generator during transmitting periods.

The ampere-hour meter integrates the withdrawal of power from the battery and starts the gas-driven generator after a drain, in this case, of 20 amp-hr. The generated current reverses direction of the meter and charging continues until the meter returns to zero. An adjustable overcharge feature is included.

Complete Networks

Based on the results of this investigation, design information is now available for system engineering complete hydrologic telemetering networks. Three such networks have been completed in the Ohio River Division. Other networks are in the planning stage and include Rough River Reservoir in Kentucky and Coralville Reservoir in Iowa. Figure 15 shows a radio installation in the Bluestone Dam telemetering network in West Virginia. Figure 16 shows the modified precipitation keying device used in the Bluestone Network. The greater capacity was necessary because of inaccessibility of the stations for four months out of each year.

CONCLUDING STATEMENT

The equipment described herein represents the present state of the art. Work is under way on design of even better keying devices, of low power equipment such as transistorized radio equipment, and of methods for telemetering temperature.

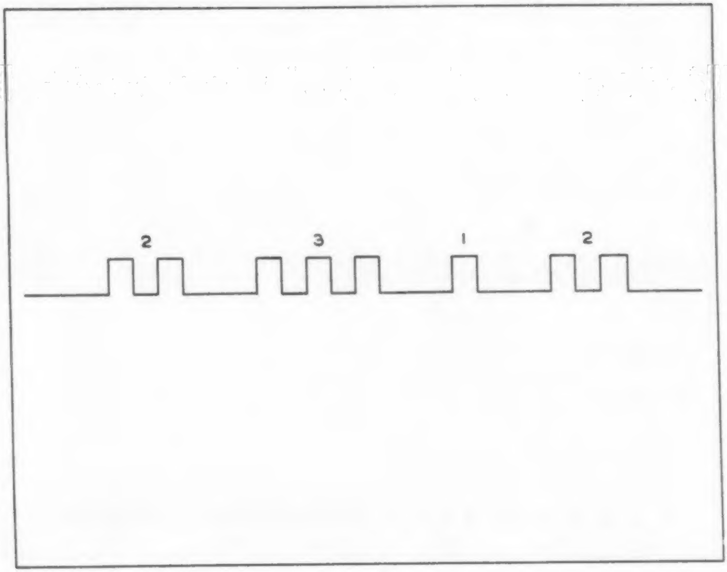

Fig. 1.--Pulse Train, Straight Digital

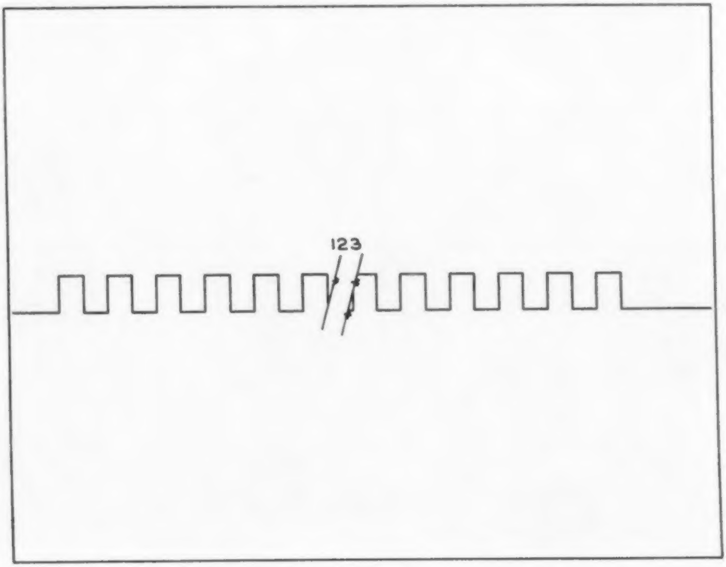

Fig. 2.--Pulse Train, Accumulative Digital

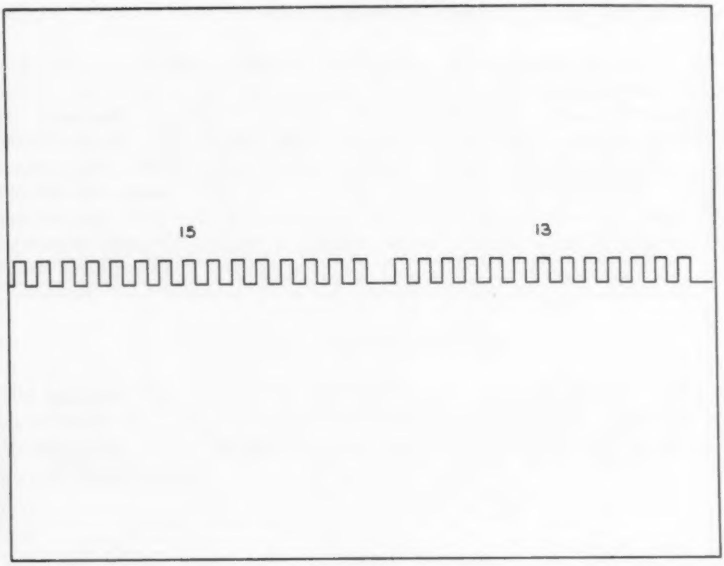

Fig. 3.--Pulse Train, Split Digital

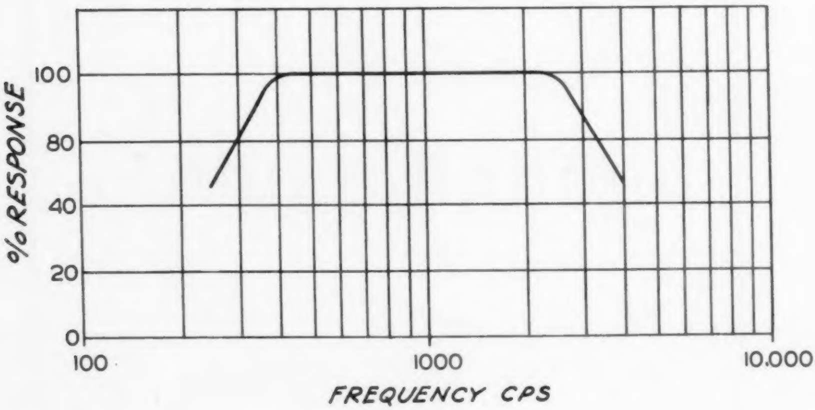

Fig 4.--Audio Pass Band of Radio Equipment

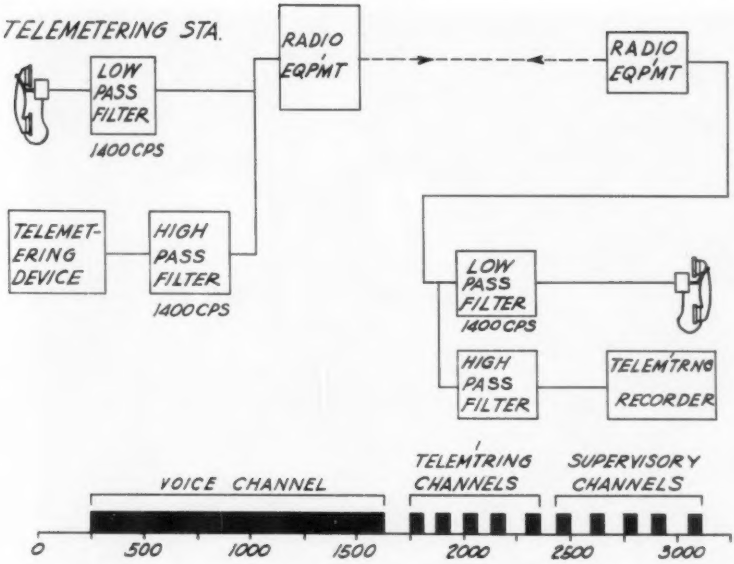

Fig. 5.--Use of Audio Spectrum

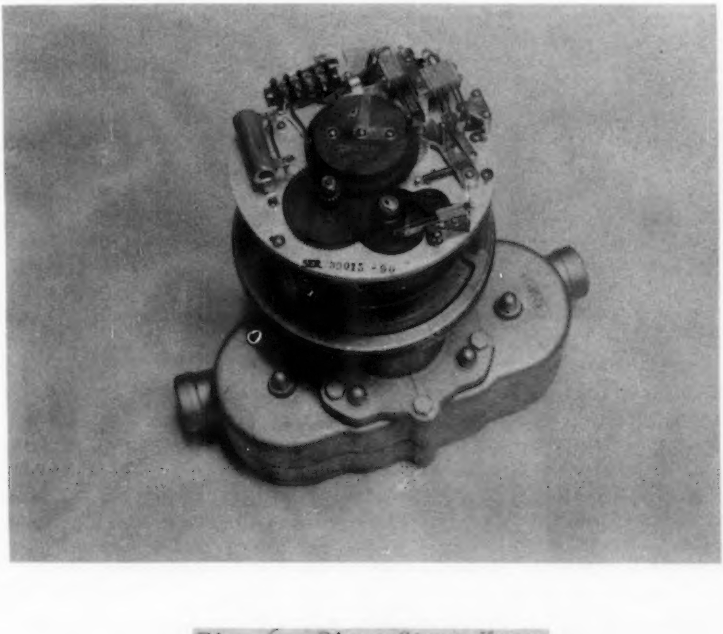

Fig. 6.--River Stage Keyer

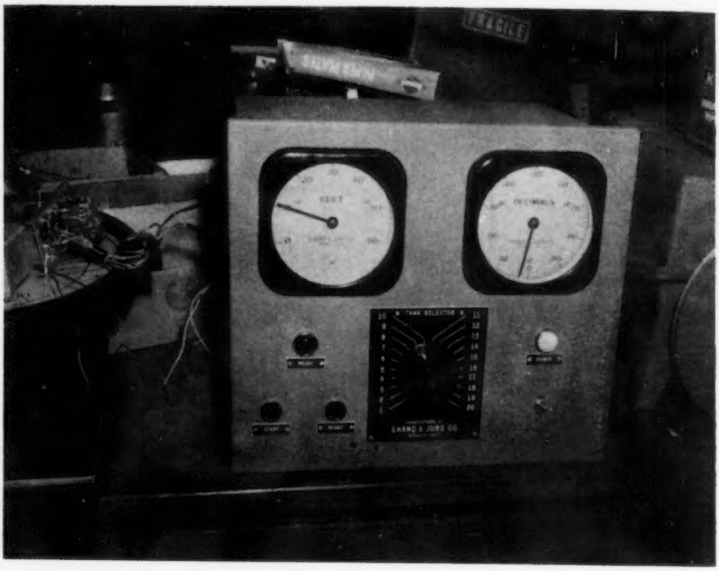

Fig. 7.--Indicator for River Stage Keyer

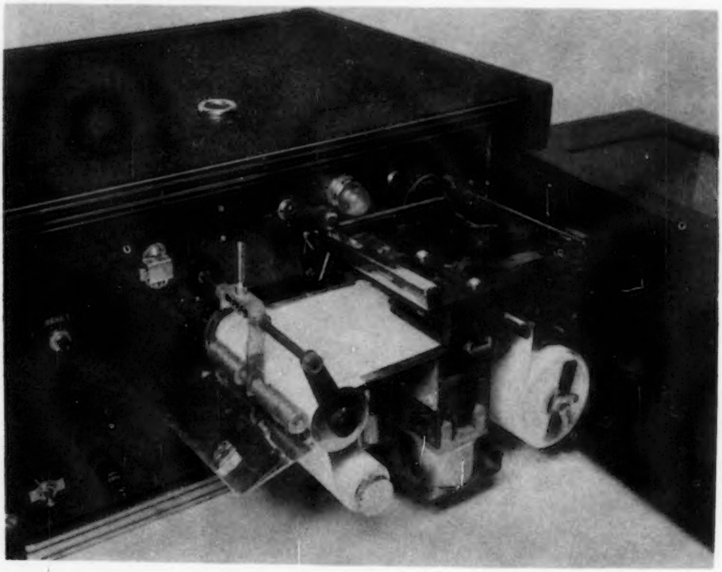

Fig. 8. Digital Print Recorder

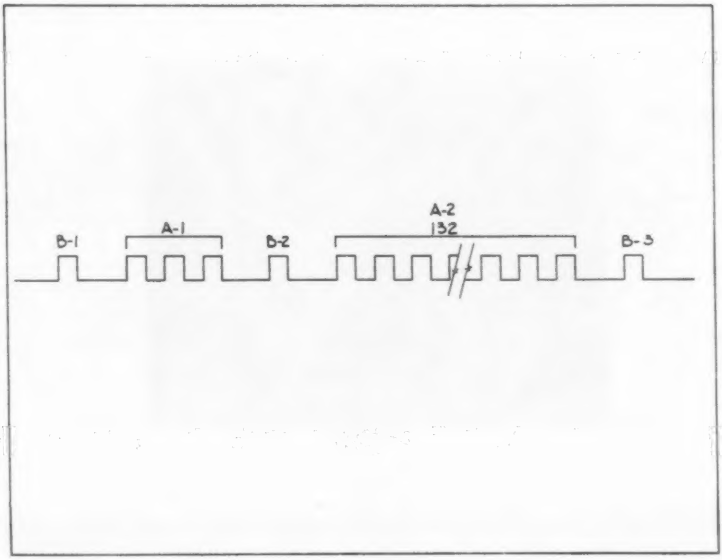

Fig. 9.--Pulse Train - Hanes Precipitation Telemetering Device

I	0	1	4	7	Dec 29 3 25 PM '55
E	0	1	4	7	Dec 29 7 51 AM '55
E	0	1	4	8	Dec 29 7 23 AM '55
B	0	0	0	0	Dec 29 7 21 AM '55
B	0	0	0	0	Dec 29 7 19 AM '55
C	0	0	0	0	Dec 29 7 10 AM '55

Fig. 10.--Sample Record from Digital Print Recorder

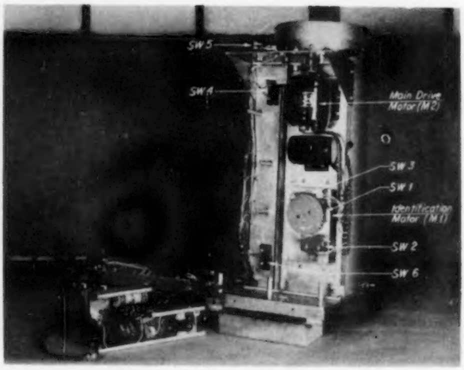

Fig. 11.--Hanes Precipitation Telemetering Device - Lettered Parts

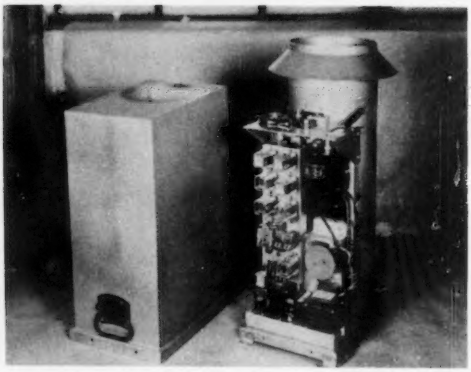

Fig. 11A.--Hanes Precipitation Telemetering Device - with Cover

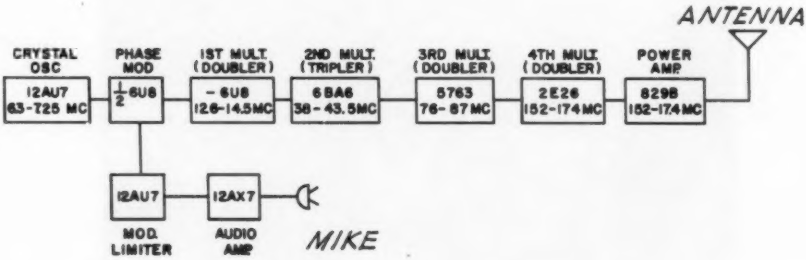

Fig. 12.--Block Diagram of Radio Transmitter

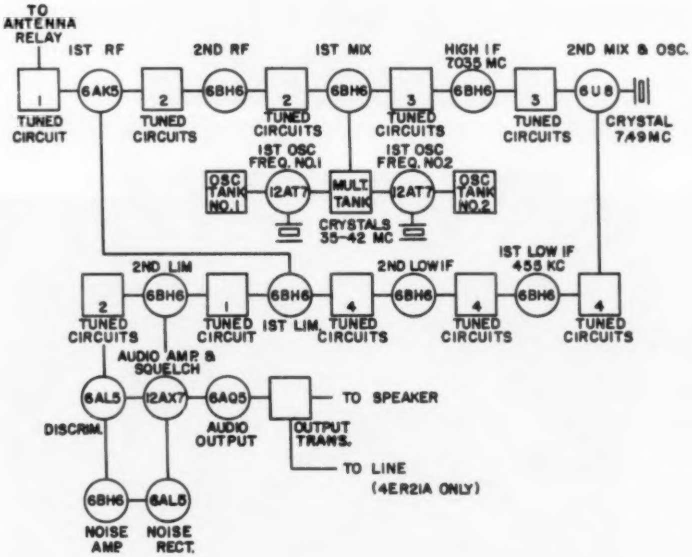

Fig. 13.--Block Diagram of Radio Receiver

Fig. 14.--Bottled-gas Operated Telemetering Station

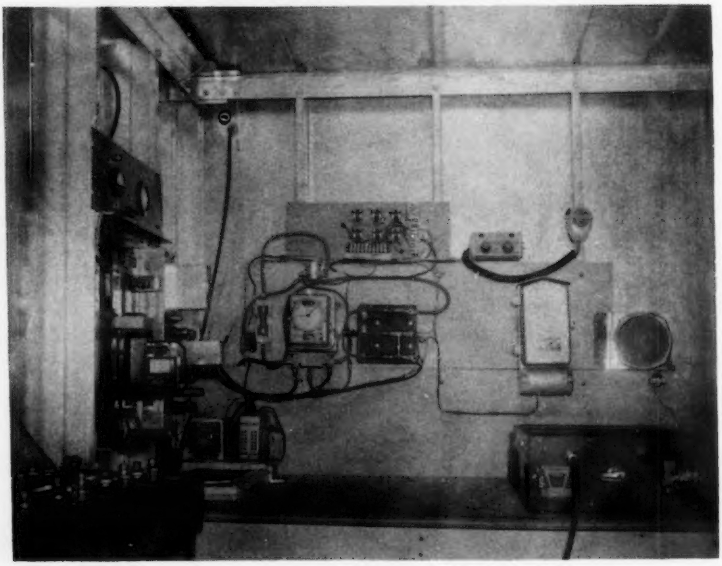

Fig. 14A.--Control Equipment in Bottled-gas Telemetering Station

Fig. 15.--Typical Radio Equipment Installation

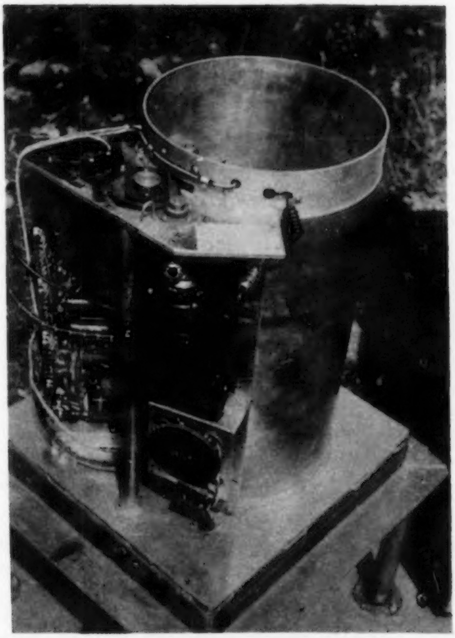

Fig. 16.--Hanes Precipitation Telemetering Device Modified for 30-inch Capacity

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CELLULAR COFFERDAMS AND DOCKS

E. M. Cummings,[1] M. ASCE
(Proc. Paper 1366)

SYNOPSIS

This paper covers a new theory of design for cellular cofferdams against failure by tilting, which is substantiated by the results of model tests. The theory gives results consistent with those obtained in the field in actual practice. Several previously published theories are discussed, and important inconsistencies are noted. Finally, the proposed theory is used to investigate the design of two actual structures, including a cofferdam on rock, and a dock in sand. Cellular docks in clay are discussed.

PART I

INTRODUCTION

The first cellular cofferdam was constructed at Black Rock Harbor, near Buffalo, New York in 1908. This was followed by the construction in 1910 of the cofferdam around the sunken Battleship Maine, in Havana Harbor. Since then cellular cofferdams have become quite common in connection with the construction of dams, locks, sewage disposal plants and other structures which cover comparatively large areas.

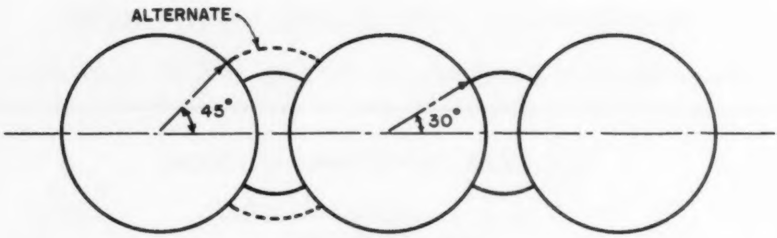
Cellular construction for permanent structures is becoming increasingly common for docks, retaining walls, breakwaters, piers, drydocks, wet berths for floating drydocks, locks, and mooring dolphins.

Types of Cellular Cofferdams

Three types of cellular cofferdams have been constructed in the past, as shown in Fig. 1. For the circular cell type, Fig. 1(a), the amount of steel sheet piling is practically constant for a cofferdam of a given depth

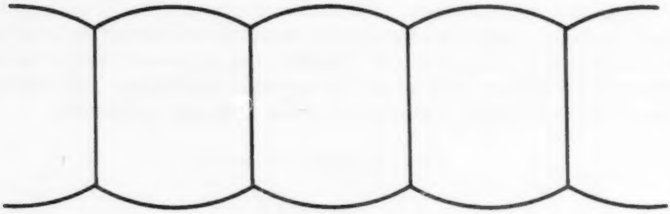
Note: Discussion open until February 1, 1958. Paper 1366 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 83, No. WW 3, September, 1957.

1. Sales Engr., Bethlehem Steel Company, Bethlehem, Pa.



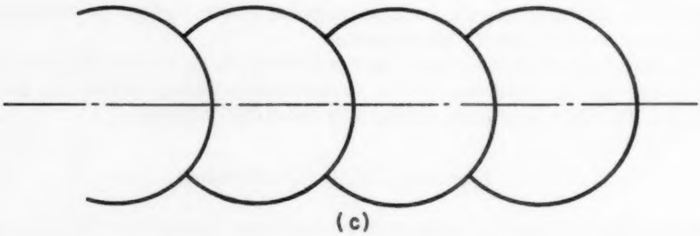
(a)

CIRCULAR CELL COFFERDAM



(b)

DIAPHRAGM CELL COFFERDAM



(c)

MODIFIED CIRCULAR CELL COFFERDAM

Fig. 1

regardless of the diameter of the cells. Because of this, for structures of more than average depth, less steel is usually required than for the diaphragm type. This often enables the designer to obtain additional factor of safety without additional cost, except for additional fill. For permanent structures such as docks, even the quantity of fill is a constant. Another advantage of the circular cell type is that each cell may be filled individually. This is of importance in rough water where storms are apt to demolish empty cells. Also, in swiftly flowing water, and in rough water, it is possible to erect successive cells by moving the pile driving equipment onto the previously filled cell. Therefore, the circular cell type is usually selected for the deepest and most difficult jobs.

The diaphragm type cofferdam of Fig. 1(b) will usually require less sheet piling for cofferdams of average or less than average height, and is considered by most contractors to be a little easier and cheaper to construct than the circular cell type in quiet water conditions. The cells should be filled in stages in order not to dislocate the cross walls. It is usual practice during filling not to permit the level of fill in one cell to vary more than about 4 or 5 feet from that in the adjacent cells. For very deep cofferdams close spacing of the crosswalls is necessary in order to keep interlock tension within permissible limits, and the sheet piling tonnage exceeds that for circular cell cofferdams.

The modified circular cell cofferdam shown in Fig. 1(c) has been used very infrequently, its advantage being that it is simpler to construct than the circular cell type, and can be filled one cell at a time. The stresses in the special piles at the junctures of the arcs must be investigated with special care. This type is not recommended for cofferdams of extreme depth.

Cellular Cofferdams on Rock

In designing cellular cofferdams on rock, three possible methods of failure must be investigated, namely sliding on the rock, bursting of the cells due to failure of the sheet pile interlocks in tension, and failure by excessive leaning or tilting of the cells due to shear failure of the fill inside the cells.

Later in this paper, sliding and interlock tension are investigated for an actual structure, and will not be covered here. The following discussion covers the possibility of failure by tilting due to lack of shear resistance of the fill.

Interior Sliding Theory of Cellular Cofferdams

This paper is based upon the theory that the stability of a cellular cofferdam against failure by tilting depends largely upon the horizontal shear resistance of the soil in the cells, with interlock friction assisting in a less important degree.

The failure of cellular cofferdams by tilting, due to horizontal sliding of the fill in the cells can be demonstrated by the following reasoning. In this discussion, it is assumed, for the moment, that interlock friction is zero, and that the cofferdam assumed depends entirely upon the fill for its stability.

Fig. 2(a) represents a cofferdam of height h and average width b, filled with sand with an angle of internal friction ϕ. Assume a condition where the

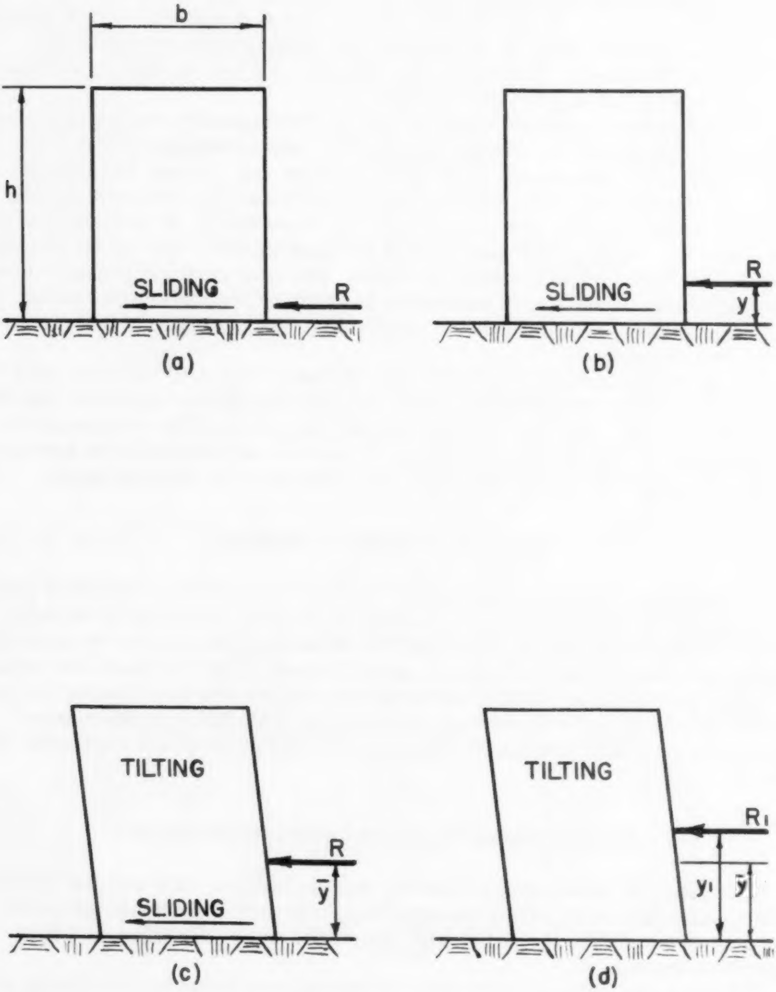

b
h
SLIDING
R
(a)
R
SLIDING
y
(b)
TILTING
R
$\bar{y}$
SLIDING
(c)
TILTING
R_1
y_1
$\bar{y}$
(d)

Fig. 2

angle of friction between the base of the cofferdam and the rock is also ϕ. Apply a force R per unit of length of the cofferdam, the point of application being very close to the base. The cofferdam will start to slide on the base when $R = \gamma h b \tan \phi$, γ being the unit weight of the fill.

Fig. 2(b) represents the same cofferdam, but now the force R is applied a short distance y above the base, so that the moment Ry is less than the moment of resistance of the cofferdam against tilting. Therefore the cofferdam will not tilt, and again it will slide on the base when $R = \gamma h b \tan \phi$.

As the point of application of the force is raised, a point will be reached where the cofferdam continues to slide and also starts to tilt. This condition is indicated in Fig. 2(c). Since we know that the force necessary to make the cofferdam slide is $R = \gamma h b \tan \phi$, we can conclude that this is also the force that makes the cofferdam tilt when applied at the distance $\bar{y}$ above the base. The moment of resistance against tilting is therefore $R\ \bar{y}$.

When a force R_1 sufficient to make the cofferdam tilt is applied at a distance $y_1 > \bar{y}$ above the base, as shown in Fig. 2(d), the cofferdam will not slide, since $R_1 < R$.

From the above, it follows that the greatest horizontal force which can be applied to a cofferdam is that which causes horizontal sliding of the fill inside the cofferdam. For a cofferdam on rock, the magnitude of this force is $R = \gamma h b \tan \phi$. The moment of resistance to tilting is $R\bar{y}$. It will be shown that this moment of resistance is constant, regardless of where the force is applied. Thus, in Fig. 2(d), $R_1 y_1 = R\bar{y}$. From this it follows that the moment of resistance against tilting remains constant for distributed loads, such as water pressure, or earth pressure.

In the following description of model tests, a theory is developed which determines the magnitude of the moment of resistance against tilting. The load measurements will show very close agreement with the theory, and the theory will indicate reasonable factors of safety for the large number of successful cofferdams which have been constructed in past years.

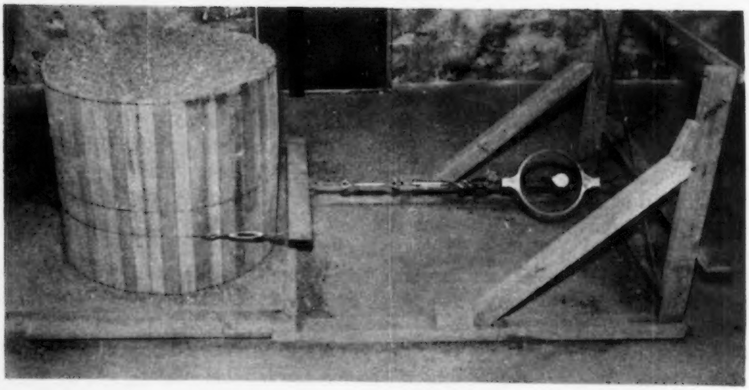
Description of Model Cell

In 1947 the writer started experimenting with model cells in an effort to develop the above theory and the model shown in Fig. 3 was constructed. Wood staves 5/16" x 1-1/2" x 2'-0" were selected to simulate sheet piles. A screw-eye was placed in the outside of each stave at 1/2", 8" and 1'-11-1/2" from the bottom, and light wire was threaded through the screw-eyes to retain the staves in the circular shape of the model cell.

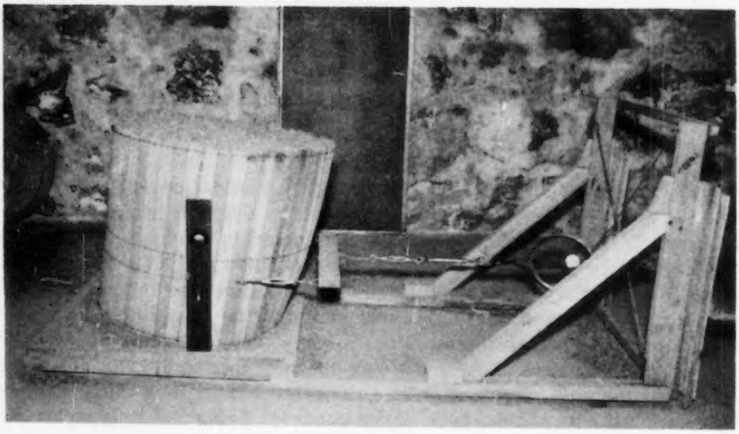
The cell rested on a platform. Preliminary tests showed that the friction between a wood platform and the fill was not adequate to develop the shear of the fill inside the cell. Consequently a concrete base 1 inch thick was placed on the wood platform, the surface of the concrete being left comparatively rough.

The staves comprising the shell of the cell were placed loosely with spaces between them up to about 1/8 inch maximum. Since they were not in contact, no interlock friction could be developed as in the steel sheet pile prototype, and the opportunity for studying the action of the fill separately was thus afforded.

The fill selected for the cell was fine crushed stone of graded sizes, the largest particle being about 1/2 inch in its greatest dimension. Stone was selected in preference to sand for two reasons, first because it did not flow



(a)



(b)

Fig. 3

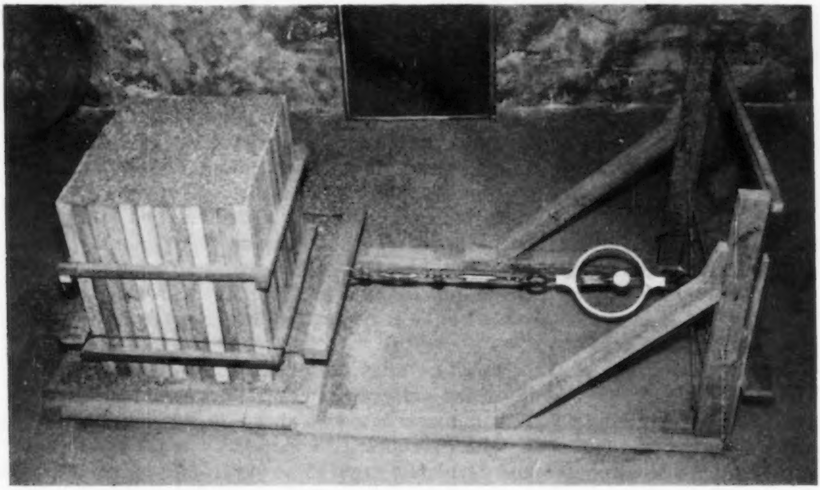


Fig. 4

Fig. 5

between the cracks of the staves and second because it was completely noncohesive. It was thought that even slight cohesion or arching developed in sand might confuse the results in a model test, while they would not be important in the full scale prototype. The fill weighed 91.9 lbs. per cu. ft.

Lateral load was applied to the cell by means of a wire placed in a half-loop around the rear of the cell. The ends of the wire extended in straight tangents at the sides of the cell and were then fastened to the ends of a small wood beam in front of the cell. See Fig. 3(a). The center of the wood beam was connected by wire to two turnbuckles in tandem which in turn were connected to a proving ring. The opposite side of the proving ring was connected to a wood beam which reacted against the knee-braced frame shown in Fig. 3. The sills of the frame extended under and were fastened to the platform on which the cell rested, making a closed system.

This set-up permitted testing of the cell with lateral load applied at any elevation from the bottom of the cell to the top. Fig. 3(b) shows the model circular cell loaded to failure with the load applied 8 inches from the bottom, which is 1/3 of the height.

The computations of stability of cellular cofferdams are customarily based upon the average width of the cofferdam. For this reason an "equivalent" rectangular model cell was constructed as shown in Fig. 4. The dimensions were $24\frac{1}{2}$ x $19\frac{1}{8}$ inches, with a height of 24 inches. The circular cell was 24 inches in diameter and 24 inches high. The horizontal cross sectional areas of the cells were approximately equal. The rectangular cell was naturally subject to much simpler theoretical analysis than the circular cell. In subsequent testing a comparison of the stability of the two cells was obtained.

Shear Resistance of Fill

From the outset of the experiment the writer was of the opinion that the fill inside a cell does not remain in an active pressure state when the cell is subjected to lateral pressure, and that the vertical shear of the fill is increased by the lateral pressure. It was decided that a logical approach would be to compare the vertical shear of the fill when the cell was first loaded laterally and then not loaded. To accomplish this, three wood test blocks of rectangular cross section were placed in the fill as shown in Fig. 6(a) for the circular cell and 7(a) for the rectangular cell. They were all cut from a smooth white pine plank, their dimensions being 3/4" x 2-1/2" x 2' -2". The width of each block was placed normal to the direction of lateral load. They were carefully placed in the empty cell and held in position as the fill was placed loosely around them, in order not to develop any unusual pressure conditions in the fill surrounding the blocks.

Using the gallows frame shown in Fig. 5, and by means of a turnbuckle and spring balance, the force required to pull each wood block was determined. Fig. 6(b) shows the pull required for each block for the circular cell, not laterally loaded, and Fig. 7(b) for the rectangular cell not loaded.

It was found that for consistent results a cell could be used for only one experiment. Therefore, after the above pull tests were completed, the cell in each case was dismantled and rebuilt, the wooden blocks being placed as before. The circular cell was then laterally loaded with 250 lbs. applied 8 inches from the base, and with this load held constant, each wooden block was pulled. The same procedure was followed for the rectangular cell, except

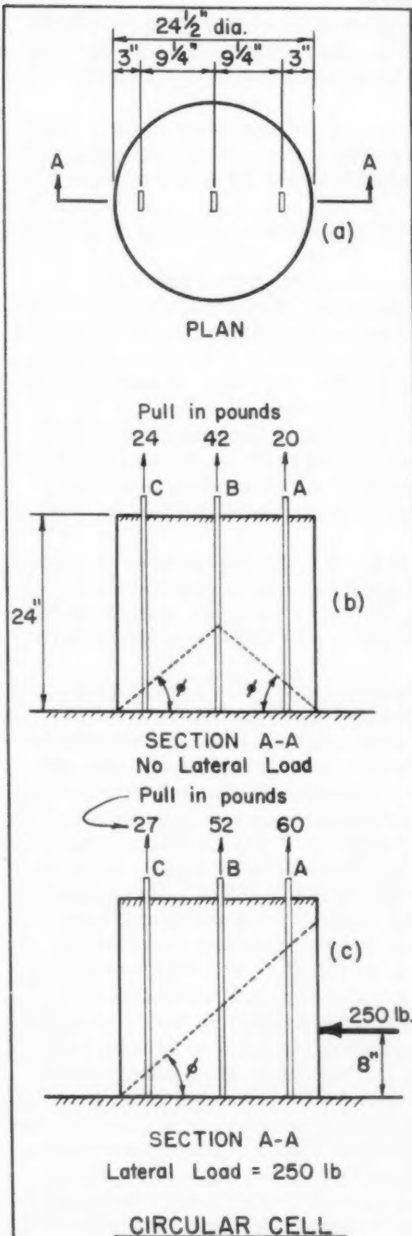


Fig 6

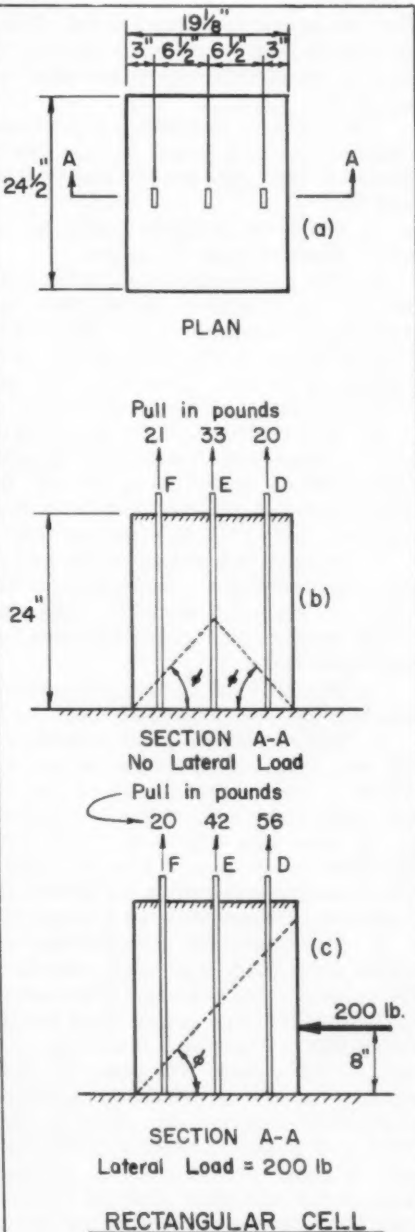


Fig 7

WOOD BLOCK PULL TESTS

that the lateral load was 200 lbs. The results of these pull tests are shown in Fig. 6(c) for the circular cell and 7(c) for the rectangular cell. Each figure for the circular cell is the result of three tests, for the rectangular cell, one test.

The following inferences may be drawn from a comparison of the forces required to pull the wood blocks when the cells were not laterally loaded as shown in Figs. 6(b) and 7(b) and then laterally loaded, as shown in Figs. 6(c) and 7(c):

1. The force required to pull the blocks is an indication of the magnitude of the vertical shear in the fill.

2. The pull required for the blocks in the center of each cell before lateral load is applied, is invariably considerably greater than for the blocks near the outside of the cell. This led to the conclusion that the fill under the ϕ-lines shown in Fig. 6(b) and 7(b) is in a different shear condition than the balance of the fill. The material beneath the "ϕ-cone" may be said to be at rest; the balance of the material is in an active pressure state.

3. Note that block A, for the circular cell, Fig. 6, required only a 20 lb. pull for the cell not loaded, but the pull increased to 60 lb. when a lateral load of 250 lb. is applied to the cell. Similarly, for the rectangular cell, Fig. 7, block D required only 20 lb. for the cell not loaded, and 56 lb. when a lateral load of 200 lbs. was applied.

4. Of equal importance is the fact that block C for the unloaded circular cell required a 24 lb. pull, while for the loaded cell there was hardly any increase, the pull being 27 lb. Likewise, for the rectangular cell, block F required 21 lb. pull for the unloaded cell, while only 20 lb. was required for the loaded cell.

5. Blocks B and E, both in the middle of the respective cells, experienced increase in shear resistance over the condition described in 2 above.

6. The conclusion is drawn from the above that the shear condition of the fill was completely transformed by the lateral pressure applied to the cell. Furthermore, increase in shear was not transmitted horizontally across the full width of the cell, since the effect of lateral pressure on one side was lost on the other side of the cell. This observation led to the conclusion that the fill above the ϕ-lines of Figs. 6(c) and 7(c) remained substantially in an active pressure state, while the portion of the fill below the ϕ-lines changed to a passive pressure state as a result of the application of the lateral load.

7. The experiment described above led the writer to conclude that the action of the fill in a cellular cofferdam is as follows. Fig. 8(a) represents the cross section of a sand-filled cellular cofferdam on a horizontal rock base, the cofferdam having tilted due to lateral pressure, as indicated by the dotted lines. The line A-B represents a plane at the angle of internal friction, ϕ, to the base. The shear resistance of the fill on plane AB is exactly equal to the tendency of the fill above to slide on the plane AB, since ϕ is the angle of internal friction. As the cofferdam tilts, A moves to A', and the new plane A' B makes an angle with the base greater than ϕ. The shear resistance on plane A' B is less than the tendency to slide on that plane. Therefore, as the cofferdam tilts, the fill above plane A' B slides down the steepened slope, within the limitations imposed by the sheet piling. The fill which lies below plane AB is transformed into a passive pressure state by the lateral pressure, and is surcharged by the fill above that plane.

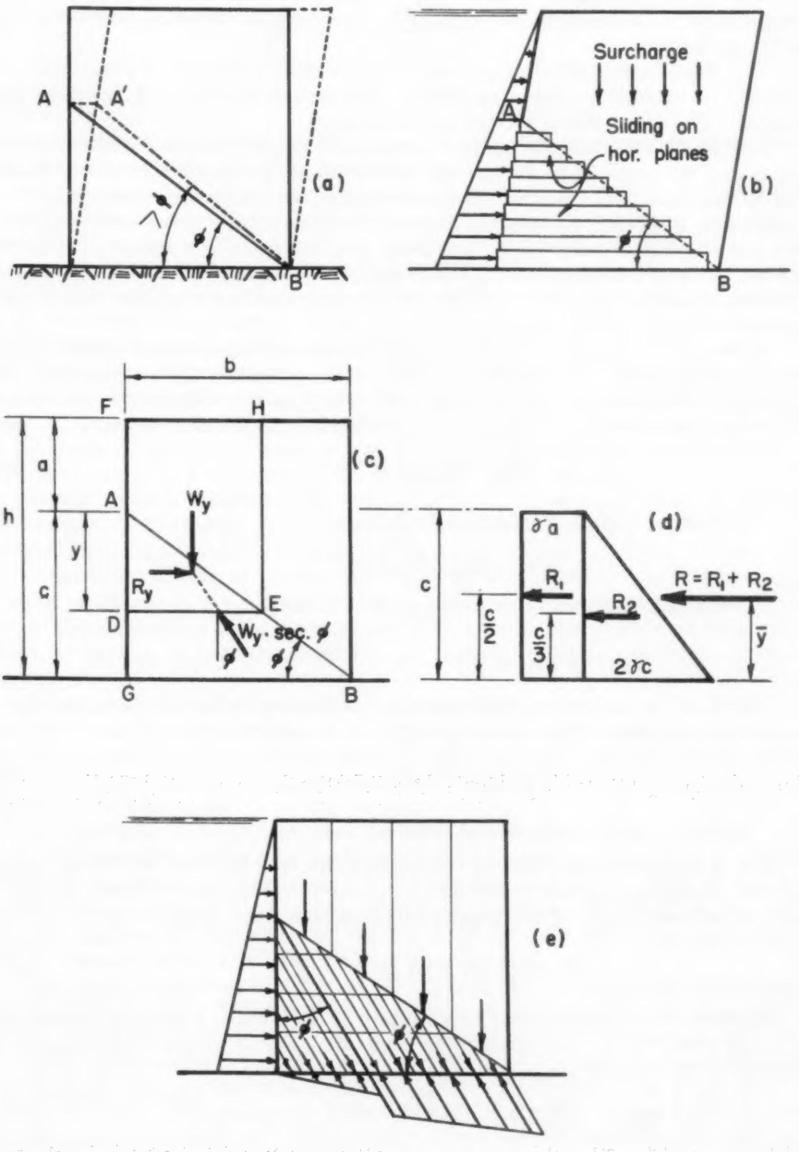

A
A'
(a)
B
Surcharge
Sliding on
hor. planes
A
(b)
B
b
F
H
(c)
a
h
A
W_y
y
c
R_y
E
D
W_y · sec. ϕ
G
B
(d)
γa
c
R₁
R₂
R = R₁ + R₂
c/2
c/3
2γc
ȳ
(e)

Fig. 8

The discussion immediately following will be in the following order: 1. Theory of resistance to failure by tilting. 2. Actual measurements of the resistance to tilting of model cells. 3. Comparison of actual measurements with the theory.

Fig. 8(b) represents diagrammatically the tilting failure of a cellular cofferdam by sliding on planes parallel to the rock base, which is assumed horizontal. The plane AB is plotted at the angle ϕ.

In Fig. 8(c) the line AB again represents a plane at angle ϕ to the horizontal base. As previously discussed, the fill above plane AB slides on AB when the cofferdam tilts. Therefore, in developing the maximum resistance of the cofferdam to tilting, no internal lateral resistance is accumulated between the top of the cofferdam at F, and point A. The sheet pile envelop, stiffened by the fill above point A, serves to transmit forces applied to the exterior of the cofferdam above point A, to the completely confined material within the prism ABG.

In Fig. 8(c) the force R_y which develops the ultimate lateral shear resistance accumulated at any depth y below A is equivalent to the resistance to sliding of the prism FHED on plane DE, which in turn is equal to the weight W_y of the prism times the tangent of the angle of internal friction, ϕ. Thus,

$$R_y = W_y \tan\phi \tag{1}$$

also

$$W_y = \gamma(a+y)y\cot\phi \tag{2}$$

where γ = the unit weight of the fill

Substituting this value of W_y in (1) we have:

$$R_y = \gamma(ay + y^2) \tag{3}$$

The force R which develops the ultimate lateral shear resistance of the entire cell is obtained by substituting y = c in equation (3). Thus,

$$R = \gamma(ac + c^2) \tag{4}$$

where $c = b\tan\phi$, and $a = h-c$.

It will be noted from this derivation that the lateral force R which develops the maximum lateral resistance is equal to the weight of the fill in the cell times the tangent of the angle of internal friction. Thus,

$$R = \gamma bh\tan\phi \tag{5}$$

Equation (4) is represented graphically by Fig. 8(d). The area of this diagram is equal to R. The moment of resistance M_s of R about the base of the cofferdam is therefore:

$$M_s = \left(R_1 \times \frac{c}{2}\right) + \left(R_2 \times \frac{c}{3}\right)$$

substituting,

$$M_s = \left(\gamma ac \times \frac{c}{2}\right) + \left(\gamma c^2 \times \frac{c}{3}\right)$$

or

$$M_s = \gamma\left(\frac{ac^2}{2} + \frac{c^3}{3}\right) \tag{6}$$

The height $\bar{y}$ of the resultant R above the base, may be found as follows:-

$$\bar{y} = \frac{M_s}{R} = \frac{M_s}{\gamma b h \tan\phi} \tag{7}$$

Testing of Model Cells

As previously discussed herein, lateral load tests were conducted on the circular and rectangular cells shown in Fig. 3 and Fig. 4.

The first tests were run on the circular cell shown in Fig. 3. The lateral load was first applied close to the bottom and gradually increased until the cell slid on the rough concrete base. It was noted that the resistance to sliding was considerably increased by the shear set up by the wooden stave shell of the model cell gripping the rough surface of the concrete. This gripping effect was eliminated by permitting the low spots in the concrete to become filled with rock dust and particles of the stone fill used in the cells. This procedure had the further advantage that the angle of friction for sliding on the base became the same as the angle of internal friction of the fill in the cell. Because of this it was possible to obtain very satisfactory values of the angle of internal friction of the fill by determining the resistance to sliding of the cell, the weight of fill in the cell being known.

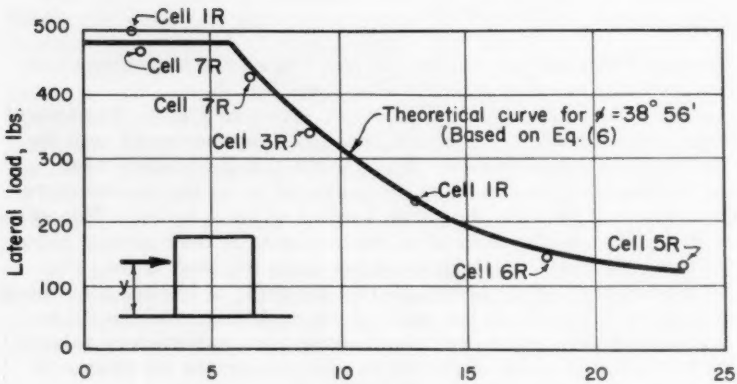
In addition to determining the amount of lateral force necessary to develop the resistance to sliding, loads were also applied 6 inches, 8 inches, 12 inches, 18 inches and 23 inches above the base. It was found that about 1-1/2 inches of tilting of the top was required to develop the ultimate resisting moment. In every case, the heel of the cell raised off the base about 1/2 inch by the time the ultimate load was reached. This raising applied, of course, only to the wood staves, the fill remaining on the concrete base.

As the cell tilted, there was a very noticeable subsidence of the fill on the toe side of the cell, and a "piling up" of the fill on the heel side. The subsidence amounted to about 1 inch. On the heel side, despite the fact that the shell raised 1/2 inch, the surface of the fill remained as high as the top of the shell of the cell.

An individual cell could be used for only one measurement of lateral load, because pulling the cell back to a vertical position after tilting involved so much manipulation of the fill that it increased in density. This resulted in turn in an increase in the angle of internal friction. The angle of internal friction increased from about 37°-40' to 42° in one cell which was tested five times, the cell being straightened up between tests.

The angle of internal friction of the fill was 38°-56', and was determined as above described. This result was in good agreement with the angle of the slope of repose, which was 36°-42'. The unit weight of the fill was 91.9 pct.

Table No. 1 gives the magnitude of the lateral load required for tilting the rectangular cell, for various heights of the point of application of the load above the base. The height y above the base is given both for the start and the conclusion of each test, the difference between the two being the amount that the heel of the cell raised during the test. The final height was used in the computations of the theoretical R_y and theoretical M_s, given in the fifth and seventh columns, respectively, of Table 1. The cell was dismantled and rebuilt for each test. Note in the 4th and 5th columns the close agreement between the actual load measured and the theoretical load computed using



COMPARISON OF THEORETICAL WITH ACTUAL RESISTANCE
(a)

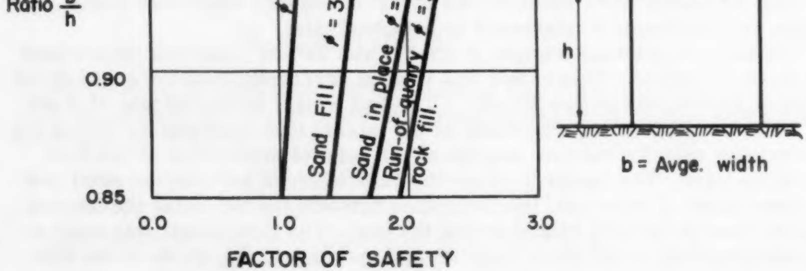


Fig. 9

TABLE 1 - LATERAL LOAD TESTS OF RECTANGULAR MODEL CELLS
COMPARISON OF TEST RESULTS WITH THEORY

Cell No.	Height y, inches: Initial inches	Final inches	Load at Failure, R_y: Actual	$\frac{M_s}{y}$	Resisting Moment at Failure, M_s: Actual ft.lb.	per Eq.6 ft.lb.	% Error
7	6	6 5/8	431	442	238	244	-2.5%
3	8	8 1/2	331	345	234	244	-4.1
1	12	12 1/2	233	234	243	244	-0.4
6	18	18 3/8	153	159	234	244	-4.1
5	23	23 5/16	133	126	258	244	+5.7
					Average Error		-1.1%

Notes: y is the distance of the point of application of the lateral load above the base. The difference between the initial and final y is due to the fact that the shell of the cell raised during testing. The final y was used in computations.

M_s is the theoretical resisting moment computed in accordance with Eq. 6.

equation (6). Note also that the resisting moment remained virtually constant for all of the points of application of the load included in the table, and that the percent variation from the theoretical computed from equation (6) was 0.4% to 5.7%, with an average error of only 1.1%.

Fig. 9(a) shows in graph form the information given in Table 1. The graph shows the theoretical lateral load plotted against the height of the point of application of the load. The actual measurements of loads for the various cells are shown. Note that the measured points are in very close proximity to the theoretical graph.

Note that for the left hand portion of the graph the theoretical resistance is constant and is equal to R of equation (5). R is the force which develops the ultimate horizontal shear resistance of the fill in the cell. The highest point at which it can be applied is the distance $\bar{y}$ given by equation (7). At any point higher than $\bar{y}$ the cell tilts from a force less than R.

Since the base under the model cells was especially prepared so that the angle of friction for sliding on the base was the same as the angle of internal friction of the fill in the cells, the value of R, the maximum lateral force that the cell could withstand, was also the resistance to sliding on the base. Note that the values for sliding for cells #7 and #1 are shown near the left hand end of the graph. The angle of internal friction, ϕ, was determined from these two values as previously explained.

Resistance to Tilting Due to Interlock Friction

An important factor adding to the resistance of a cellular cofferdam against failure by tilting is interlock friction. As an aid to estimating the magnitude of this resistance, it is convenient to assume a cofferdam composed of rectangular cells with straight crosswalls uniformly spaced at

any distance, L. The width of the cofferdam is b, equal to the average width of cofferdam used in deriving equations (1) to (7).

It is customary to assume that the active pressure condition of the fill is

$$P = 1/2\, \gamma h^2 \tan^2 \left(45^\circ - \frac{\phi}{2}\right) \tag{8}$$

where P is the total pressure per running foot of cofferdam for the total depth h. The interlock tension in the crosswall is then PL. Assuming a coefficient of friction f in the interlocks, the friction in the interlocks is PLf. If b_1 is the width of one pile, then the couple $PLfb_1$ is the moment of resistance of one pile of the crosswall against tilting. If there are n piles in the crosswall, the total resistance of the crosswall to tilting becomes $PLfnb_1$. But nb_1 equals the average width b of the cofferdam. Therefore the total moment of resistance to tilting of the crosswall is PLfb. Since the crosswalls are uniformly spaced at distance L, the resisting moment due to interlock friction, per foot of cofferdam, is

$$M_f = Pfb \tag{9}$$

Combining equations (6) and (9) we have the total moment of resistance of a cellular cofferdam filled with sand, gravel, sand and gravel, or crushed rock, and resting on a horizontal rock base. It is

$$M_r = \gamma\left(\frac{ac^2}{2} + \frac{c^3}{3}\right) + Pfb \tag{10}$$

where $c = b \tan \phi$ and $a = b-c$.

Many precedents have established the practice of designing cellular cofferdams on rock so that the average width is not less than 0.85 of the height, or $\frac{b}{h} = 0.85$. The average width b of a diaphragm type cofferdam (Fig. 1(b)) is computed by dividing the area enclosed by the cell by the distance between crosswalls. For the circular cell type (Fig. 1(a)) the area of one cell is added to the area between cells and the sum is divided by the distance center to center of cells.

The pressure of fresh water per linear foot of cofferdam is

$$\frac{1}{2} \times 62.5 h^2 = 31.25 h^2$$

The active moment of water pressure is then

$$M_w = 31.25 h^2 \times \frac{h}{3} = 10.42 h^3$$

The factor of safety against tilting due to water pressure is obtained by dividing the moment of resistance from Eq. (10) by the active moment of the water pressure. Whence,

$$\text{Factor of safety} = \frac{M_r}{M_w} \tag{11}$$

If the angle of internal friction of the fill or of the soil in place at the site is not determined by laboratory methods, the following values are suggested.

	ϕ-deg.	γ-lb/cf
Sand fill	30	100
Sand in place	35	105
rock fill (run of quarry)	38	100

Table 2. Values of C tan ∅ - using Krynine's value of C

∅ - degrees	0	10	20	30	40	50	60	70	80	90
C tan ∅	0	.17	.29	.35	.35	.31	.25	.17	.09	0

Table 3. Factor of Safety using Krynine's value of C

∅ - degrees	0	10	20	30	40	50	60	70	80	90
G_s	.82	1.22	1.43	1.43	1.28	1.06	.79	.51	.24	0

Table 4. Various values of ∅ substituted in Anderson's problem

∅ - degrees	50	40	30	20	10	0
Average width b - feet	106	85	72	65	63	70

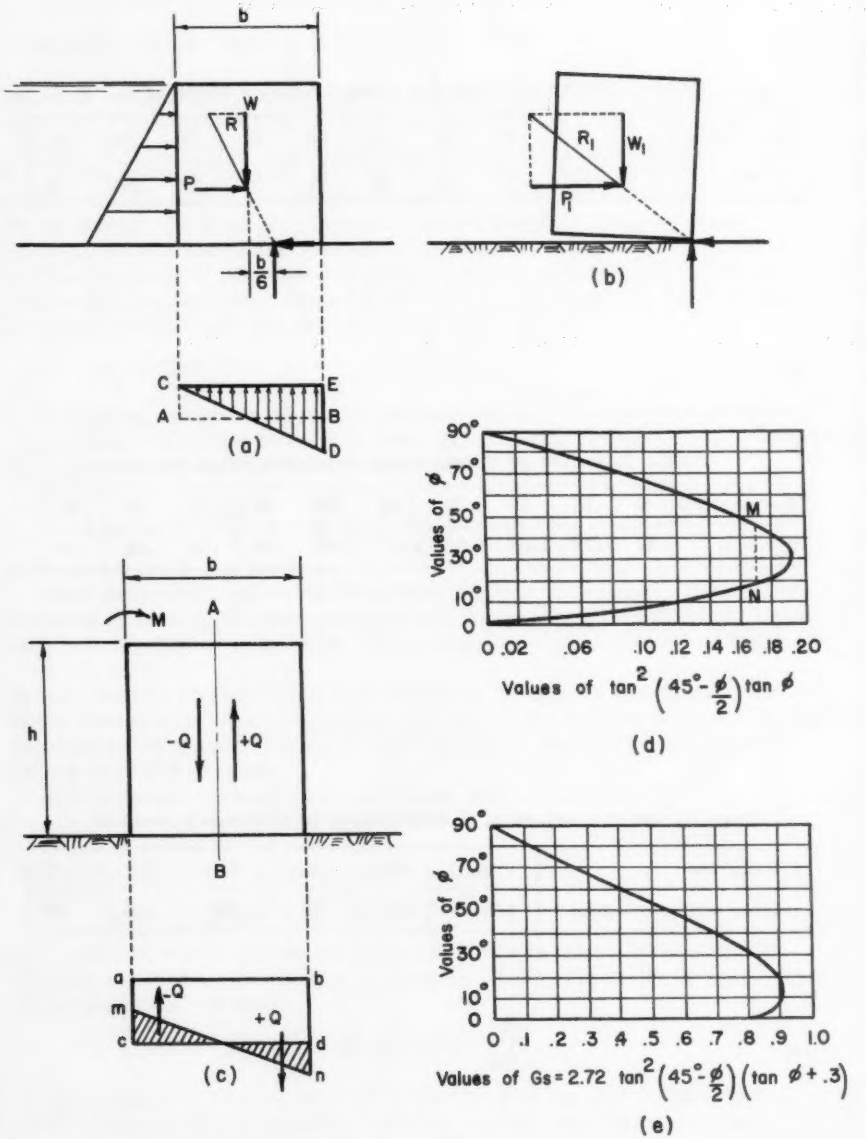

b
W
R
P
b/6
C
E
A
B
D
(a)
R1
W1
P1
(b)
M
A
b
h
-Q
+Q
B
a
b
m
-Q
+Q
c
d
n
(c)
90°
70°
50°
30°
10°
0
Values of ø
M
N
0 .02 .06 .10 .12 .14 .16 .18 .20
Values of tan² (45° - ø/2) tan ø
(d)
90°
70°
50°
30°
10°
0
Values of ø
0 .1 .2 .3 .4 .5 .6 .7 .8 .9 1.0
Values of Gs = 2.72 tan² (45° - ø/2) (tan ø + .3)
(e)

Fig. 10

Fig. 9(b) shows the factor of safety against tilting due to the pressure of fresh water, for each of the above, for values of the ratio $\frac{b}{h}$ from 0.85 to 1.00.

Discussion of Previously Published Methods of Design

The original designers of cellular cofferdams, between 1908 and 1923, investigated the possibility of overturning in a manner similar to that used for mass retaining walls. In fact, this method is used by many designers to this day. In Fig. 10(a) the resultant R of the weight W of the fill and the horizontal thrust P was required to intersect the base within the "middle third". When this precaution was taken, the assumption was that the factor of safety against overturning was at least 3.0. When the resultant is at the edge of the middle third as shown in Fig. 10(a), the base pressure, according to this theory, would be represented by the triangle CED, the rectangle ACEB representing the uniformly distributed base pressure before the application of the lateral load. For the condition shown, the heel of the cofferdam, including the fill, is about to lift off the rock base. The compression on the base to the left of the center line is less than the previously uniformly distributed base pressure, and is zero at the heel. This decrease in base pressure can be accomplished only by tension in the fill to the left of the center line. Since sand, which is commonly used for fill in cellular cofferdams, is essentially non cohesive, and can develop tension in only slight degree, if at all, it follows that this assumption of the base pressure is incorrect.

Carrying the reasoning one step further, the cofferdam would still have a factor of safety of 1.0 and not be on the verge of failure until it is loaded as shown in Fig. 10(b). The heel of the cofferdam is shown raised slightly off the rock base, and the base pressure is now represented by a single vertical force and a horizontal force both applied at the very toe of the cofferdam. There is, of course, no possibility that this could happen.

The overturning theory, while based on a fallacy, happens to produce satisfactory results where sand, or gravel and sand fill is used on a rock base if the middle-third requirement is adhered to. The factor of safety, however, is not 3 when this procedure is followed. The model cells described earlier in this paper failed under a lateral load only 1.40 times the load which caused the resultant to intersect the base at the edge of the middle third.

The weakness of the overturning theory is that it does not take into account the shear resistance of the fill. There have been several notable failures and near failures which have occurred because of the use of clay fill, or because of the existance of soft clay at the site, where failure resulted from deficient shear resistance of the soil. The middle-third requirement in these cases did not prevent failure.

Another theory[3a] is based upon the assumption that the maximum internal shear occurs along the center line AB, Fig. 11(c), and that the cofferdam will fail when this shear becomes excessive. The shaded triangles in the lower sketch of Fig. 10(c) represent the stresses produced on the base by the overturning moment M. If Q is the area of one of the triangles, then M = 2/3 bQ, or Q = 3M/2b. The shearing force on plane AB is Q, and is equal to the product of the earth pressure and the coefficient of internal friction, tan ϕ. The shear on plane AB is thus

$$S = \frac{1}{2} \gamma h^2 C \tan \phi \tag{11}$$

where C is the coefficient of pressure. If the Rankine coefficient $\tan^2 (45 - \phi/2)$ is substituted for C, the equation becomes

$$S = \frac{1}{2} \gamma h^2 \tan^2 (45 - \frac{\phi}{2}) \tan \phi \qquad (12)$$

For any given cofferdam $1/2 \gamma h^2$ is a constant. In Fig. 10(d) the values of the balance of the expression, namely $\tan^2 (45^0 - \phi/2) \tan \phi$, are plotted as abscissae against values of ϕ as ordinates. The curve indicates that the maximum shear on plane AB occurs when $\phi = 30^0$. Selected stone, with $\phi = 45^0$ (point M) would be no better than poor material with $\phi = 17^0$ (point N). For material with $\phi = 90^0$ (an imaginary soil with infinite shear resistance since $\tan \phi$ is infinity), the shear, and thus the resisting moment against tilting, would be zero, as though the cells were left filled with water.

Fig. 10(e) shows the factors of safety based on this theory for a cofferdam with the ratio of average width to height of 0.85, a proportion which has been used successfully. The curve is plotted for values of ϕ from 0 to 90^0. The maximum factor of safety is only 0.9, for $\phi = 15^0$. As the quality of the fill improves, the factor of safety decreases, becoming zero for $\phi = 90^0$.

Another author (3e), discussing the above theory, suggests a value for C of

$$C = \frac{\cos^2 \phi}{2 - \cos^2 \phi} \qquad (13)$$

This value of C gives a maximum factor of safety for the problem of Fig. 10(e) of 1.43 when $\phi = 30^0$, and the same factor when $\phi = 20^0$. As the quality of the fill improves, the factor decreases, again becoming zero when $\phi = 90^0$.

When $\phi = 30^0$ is substituted in Eq. (13), the value of C is 0.60, almost double the Rankine coefficient of pressure. Interlock tension is commonly computed using the Rankine coefficient. Doubling the pressure by using Eq. (13) is not in accord with usual practice.

Application of Theory to Actual Field Conditions

Actual field conditions vary somewhat from the theoretical concept of a cofferdam previously discussed. For instance, the surface of the fill in the cells is oftentimes several feet below the top of the cofferdam. Another factor is that for a cofferdam on rock, the fill is saturated for some distance above the rock, the depth of saturation depending upon the porosity of the fill, the roughness of the rock and the provisions for drainage of the fill.

The cellular cofferdam for the Pickwick Landing Lock on the Tennessee River affords a good opportunity to test the theory presented herein, taking into account several of the variables that occurred during the life of the cofferdam. The cofferdam was of the type shown in Fig. 1(a), the main cells being 59.89' diameter, on 61.66' centers. Average width was 47.3'. Fig. 11(a) shows a cross section of the cofferdam. Assuming complete drainage of the fill, with ϕ for dry sand fill = 30^0, $\gamma = 100$ lb per cu. ft., the moment of resistance against tilting is given in Eq. (10) and is 2,054 Kip ft.

Fig. 11(a) shows the river near flood stage, close to the top of the cofferdam. The moment of the water pressure about the base was then 1,464 Kip ft.

Factor of safety is $2054 \div 1464 = 1.40$

Mr. R. T. Colburn (3c) published complete information on this cofferdam, including curves showing the saturation level inside the cells for various stages of the river. Fig. 11(b) shows, approximately, one curve obtained when the river started to fall after reaching flood stage. A portion of the fill is submerged, and a direct application of Eq. (10) is not feasible. The resisting moment of the fill is therefore computed on the basis of increments using the theory upon which Eq. (6) is based. For instance, to obtain the increment between A and B, determine the difference between the weights of the prisms OACE and OBDF. The amount of the increment is this difference multiplied by tan ϕ. The other increments are figured similarly. The total resisting moment of the fill is then the sum of the products of the increments and their respective distances above the rock base. The weight of the submerged fill is taken as 60 lbs. per C.F., and of the dry fill 100 lbs. per C. F.

Another important source of resisting moment results from the differential water level inside the cofferdams. The pressure against the inside of the wall on the river side is equal to the active pressure of submerged earth plus the full head of water. The pressure against the inside of the wall on the dewatered side is the earth pressure plus the pressure due to a considerably reduced head of water. These pressures are shown in Fig. 11(b). It is readily seen that these unbalanced pressures inside the cofferdam add to the stability against tilting.

Another source of resisting moment under this partially saturated condition is the increased interlock tension due to the water pressure which is added to the earth pressure against the inner wall of the cofferdam. This pressure is shown in Fig. 11(b), and produced a resisting moment of 903 kip ft.

Taking into account all of the above factors, the total resisting moment against tilting becomes 2987 kip ft.

At this stage the water pressure due to 52 feet of water was 84.4 kips, with a moment of 1464 kip ft.

Factor of safety is $2987 \div 1464 = 2.04$.

The above indicates a factor of safety due to the differential water level greater than that for the cells assumed completely drained.

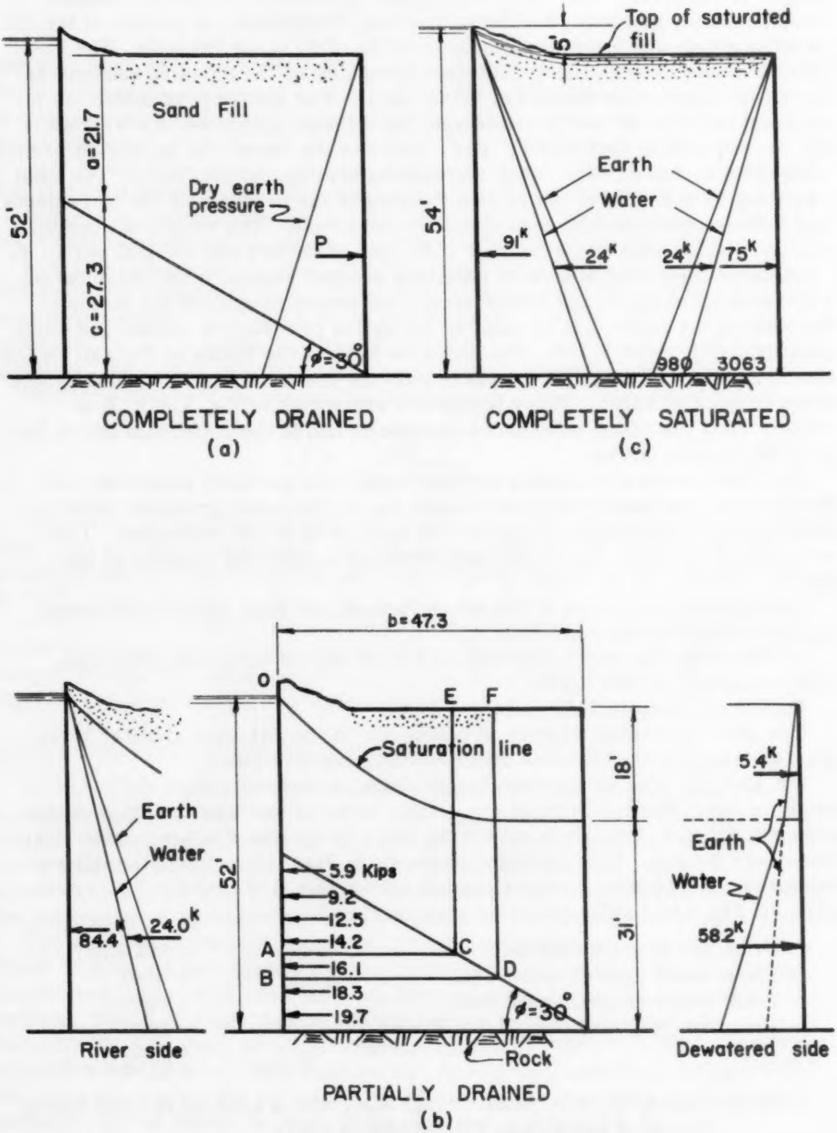
Mr. Colburn also states that the cofferdam withstood a head of the full height of the cofferdam without any visible signs of distress, and that at this point the fill was completely saturated, since there was discharge from weep holes near the top. This condition is shown in Fig. 11(c). Since the fill was saturated, its resisting moment against tilting was .6 of that for the dry condition of Fig. 11(a). The resisting moment then becomes:-

From fill in cells 1486 x .6		+ 892 Kip ft.
From water against outer wall		+ 1640
From water against inner wall		- 1226
From interlock friction 99.0 x .3 x 47.3		+ 1404
	Total	+ 2710 Kip ft.

Water pressure for 54 ft. head = 91125 lbs., with a moment of 1640 Kip ft.

Factor of safety was $2710 \div 1640 = 1.65$

Again, the factor against tilting is adequate, confirming Mr. Colburn's observation concerning no visible signs of distress. In this instance however, the stability is obtained at the expense of rather high interlock stress as discussed below.



PARTIALLY DRAINED
(b)

PICKWICK LANDING COFFERDAM

Fig 11

Interlock Tension

The equation for unit interlock tension is $T = \rho R$, where ρ is the unit pressure against the sheet piling and R is the radius on which the sheet piling is driven. In the problem under discussion, the greatest interlock tension occurred when the fill was completely saturated, the condition shown in Fig. 11(c). The greatest unit pressure was at the bottom of the inside wall, and the interlock tension according to the above formula was

$$T = 4043 \times 29.95 = 120{,}800 \text{ lbs. per ft. or } 10{,}067 \text{ lbs. per inch}$$

The sheet piling used had a guaranteed interlock strength of 16000 lbs. per inch. The factor of safety was therefore 1.60. It is advisable if possible to hold interlock stress down to a factor of safety of about 2.0. There is always the possibility that stresses will be set up in the interlocks during driving, particularly where there is a considerable depth of hard over-burden. Adequate provision for drainage of the cells is desirable in order to lower the saturation level in the cells, and thereby reduce the interlock tension.

Sliding

There is no record that has come to the notice of this writer of a cellular cofferdam on a rock foundation that has failed by sliding. Perhaps the best explanation of this is the grip that the steel sheet piling has on the rock surface. If the surface of the rock is soft, the steel will penetrate a few inches and add greatly to the resistance against sliding. Even where the rock surface is hard, natural irregularities in the rock surface provide adequate resistance against sliding. In so far as the sliding resistance of the fill on the rock surface is concerned, the coefficient of friction cannot exceed tan ϕ of the sand for if it were greater, a lesser resistance to sliding would exist in the sand immediately above the rock. For the Pickwick cofferdam the least resistance to sliding occurred for the condition of Fig. 11(c) where the fill was completely saturated. Using the unit values previously assumed the resistance to sliding of the fill on the rock was $bh\gamma \tan \phi = 80300$ lb. Water pressure against the cofferdam was 91,125 lb. for 54 ft. head.

This computation indicates that the water pressure against the cofferdam exceeded the sliding resistance of the fill, and that the grip of the steel on the rock was the determining factor preventing sliding. The importance of drainage of the fill is again emphasized, since if the fill had been completely drained as in Fig. 11(a), or partially as in Fig. 11(b), the resistance to sliding would have been increased due to the increased weight of the fill.

Cofferdams on Foundations other than Rock

While most cellular cofferdams in recent years have been constructed on rock foundations, there have been many cofferdams constructed on sand foundations, notably for the locks and dams on the Mississippi River. Here the chief problem was seepage under the cofferdam through the sand. This type of cofferdam has been covered at length by White and Prentis in a volume entitled "Cofferdams" (6). Examples of cofferdams of this type are shown in Booklet 127-C, "Typical Installations of Steel Sheet Piling" (7) published by Bethlehem Steel Company. In this type of cofferdam a sand berm is placed against the inner face of the cofferdam for two purposes, first to force the "flow net" of the seepage water away from the toe of the cofferdam,

in order to prevent upward flowing water from making the sand foundation "quick" adjacent to the toe of the cofferdam, and secondly to add to the stability against tilting.

Cellular cofferdams on clay foundation are rare. The writer knows of one large structure built about 30 years ago that started to fail by tilting and excessive base pressure at the toe, when the water had been pumped down only part way. It was stabilized by a heavy sand berm against the inner face. In this case, the soil enclosed by the cells was original clay of deficient shear value.

Cellular cofferdams have been successfully constructed in very soft clay where resistance against tilting has been provided by other means. For instance, in 1932 an extremely large cofferdam was constructed in the Hudson River in New York between 48th and 52nd Streets. Here the river bottom is very soft clay for great depths, underlain by hardpan over rock. Stability against tilting was obtained with a heavy rock fill berm against the inner face of the cofferdam.

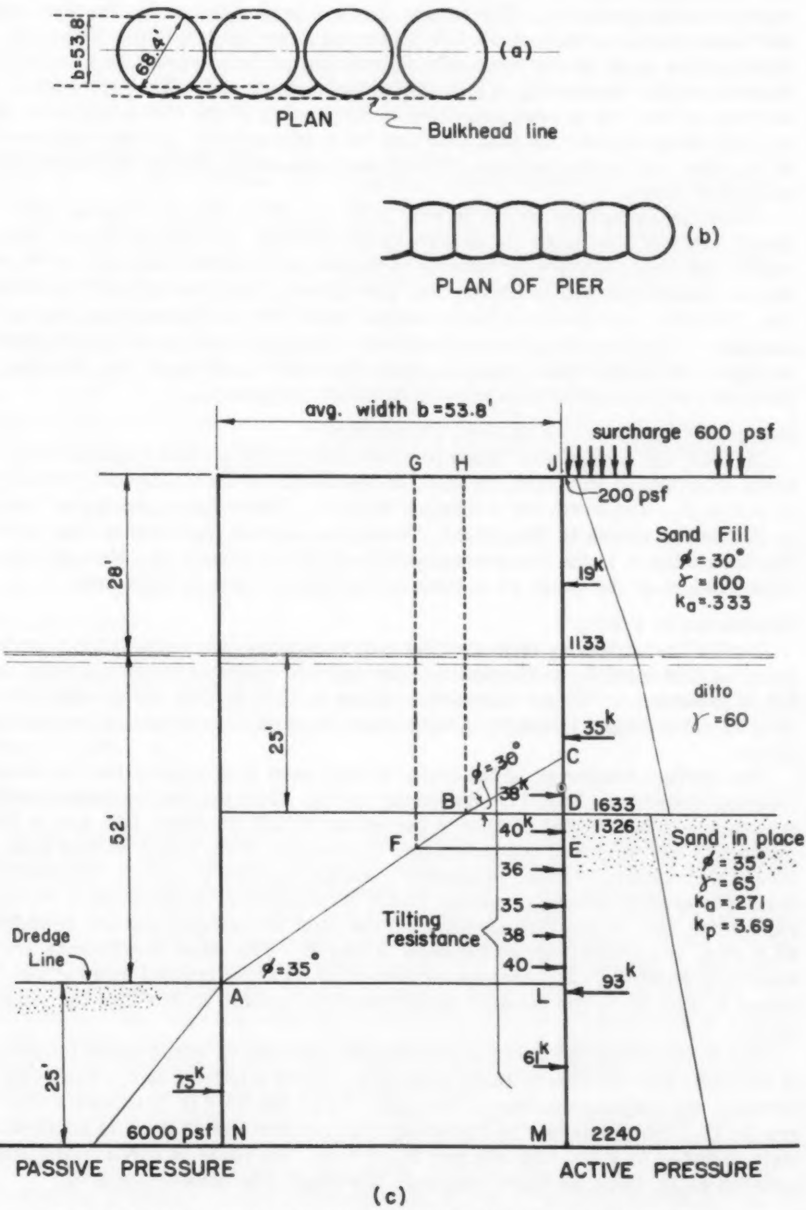
In general, it may be stated that the consistency of clay at a site must be of the stiff or hard categories unless some such precaution is taken.

Cellular Docks and Piers

Cellular construction, besides being used for cofferdams, is also used for such structures as docks, piers, breakwaters, mooring dolphins and retaining walls. Where soil conditions are appropriate, permanent structures of this type possess a high degree of stability. In addition to its ability to resist static forces, cellular construction can absorb such forces as wave action, ice and the normal impact of ships.

Cellular construction is considered for docks where the depth of water required exceeds the range of depth for which a sheet pile bulkhead is feasible, or where bed rock occurs at an elevation which does not permit sufficient penetration for a sheet pile bulkhead. Conditions are suitable where the cells rest on bare rock or on rock with an overburden of firm soil such as sand, sand and gravel, or stiff clay, or where the cells are driven into such firm soils. If the rock or firm soil is overlain by mud or soft clay, the poor soil must be removed either by predredging the site or by excavating inside the cells before filling. It is of the greatest importance that the cells rest on a base of hard or firm material that possesses the bearing capacity to sustain the weight of the filled cells, and that the original soil and the fill in the cells have adequate shear value to resist the lateral pressure against the cells. This writer knows of several very bad failures of cellular construction where the cells were driven into soft soil. In such cases placing good fill on top of mud or very soft clay does not result in material of good shear resistance. Usually the heavy sand fill will sink into the soft material and lose its separate identity, the result being a mixture of the sand and soft material which has no better shear resistance than the original soft soil.

Fig. 12(a) shows a plan of a circular cell type of cellular dock. Only one connecting arc is required where there is earth fill on one side. For piers and breakwaters where there is water on both sides, the second connecting arc is necessary. One important precaution should be taken in designing the connecting arc for a dock where a deck with a straight edge is required. It



CELLULAR DOCK - SWAN ISLAND - PORTLAND, ORE.

Fig.12

should not be designed so that it is tangent to the dock line, but about 2 feet back from the dock line. The reason for this is that the main circular cells increase their diameter about 1-1/2 percent when the pressure of the fill takes up the slack in the sheet pile interlocks, with the result that the chord distance of the connecting arc is appreciably shortened. Since the middle ordinate of the arc is very sensitive to shortening of the chord distance, the arc will bulge beyond the bulkhead line very appreciably, making construction of the deck and fender system difficult and expensive, unless the above precaution is taken.

The diaphragm type shown in Fig. 11(b) is also suitable for docks and piers, and has been used for numerous structures. For docks in very deep water, the cross walls may have to be spaced so close to keep the interlock stress within reasonable limits, that the circular cell type is more economical. However, for docks of intermediate depth the diaphragm type may be the cheaper. Furthermore, in cases where a concrete deck must be constructed on top of the cells, with a straight dock edge on the bulkhead line, the diaphragm type may present fewer construction problems.

Design of Cellular Dock on Sand Foundation

In 1943, the writer was asked to cooperate with the chief engineer of a large shipyard at Portland, Oregon, in the design of a cellular dock which was to serve as a wet berth for a floating drydock. The deepest portion of the dock is of the type shown in Fig. 12(a). The cross section is shown in Fig. 12(c), the dimension b being the average width of the circular cell. Note the extreme height of the dock, which was 80 feet from top to dredge line.

Resistance to Tilting

The following theory of design for safety against failure by tilting, is the same as that used for cofferdams. The angle of internal friction for the sand fill is assumed at 30°, for the sand in place at 35°. In Fig. 12(c), ABC, the ϕ line, is not straight, because of the change in ϕ at the surface of the sand in place.

The method employed is the same as that used in analyzing the Pickwick Landing cofferdam, Fig. 11. Referring to Fig. 12(c) the top increment is the resistance to horizontal sliding of the prism HJDB on plane BD, and is the product of the weight of the prism and the tangent of ϕ, which in this case is 30°. The product, 38 kips, is applied midway between C and D. Similarly, the resistance to sliding of prism GJEF on plane FE is obtained, ϕ in this case being 35°. From this is deducted the first increment, and the remainder, 40.4 kips, is applied midway between D and E. The other increments are similarly obtained. The bottom increment of 61 kips, applied midway between L and M, is the product of the weight of prism ALMN and the tangent of ϕ.

The moments of the various increments are then obtained about the bottom of the cell, and the sum of their moments, 10,057 Kip feet, is the resisting moment per longitudinal foot of the cell. Since the 68.4 ft. diameter cells are on 73.1 foot centers, the resisting moment per foot of dock is proportionately reduced to 9,400 Kip feet per ft. of dock. No value is given to the soil between cells, because there was only one sheet pile connecting arc.

Resistance due to Interlock Friction

As in the case of cofferdams, interlock friction adds to the resistance to tilting. In this case, the active pressure without surcharge is taken only

down to the dredge line, since the passive pressure of the fill below that elevation will prevent tension in the interlocks. The active pressure is readily determined from Fig. 12(c) by deducting the pressure due to surcharge. The resulting pressure P, is 81 Kips per foot of dock. The resisting moment is Pfb = 1306 Kip ft. per ft. of dock, where f is the coefficient of interlock friction.

Passive Pressure Moment

Tilting is further resisted by the passive pressure of the soil against the outboard face of the cell. For $\phi = 35^{\circ}$, the coefficient of passive pressure k_p is 3.69. The intensity of passive pressure at El. -77 is therefore 25 x 65 x 3.69 = 6000 psf. Total passive pressure is 75 Kips applied at the centroid of the diagram. Resisting moment about El. -77.0 is 623 Kip ft.

Active Pressure Moment

The active pressure diagram of Fig. 12(c) is obtained by using a coefficient of pressure for sand fill of .333, corresponding to an angle of internal friction $\phi = 30^{\circ}$. For sand in place the coefficient is .271, for $\phi = 35^{\circ}$. Total pressure applied at the centroid is shown in Fig. 12(b) for each portion of the pressure diagram. Moment of the active pressure about the bottom of the cell at El. -77.0 is 6098 Kip ft.

Factor of Safety Against Tilting

The sum of the resisting moments is 11,329 kip ft. The active moment is 6098 kip ft., and the factor of safety against tilting 1.86.

Factor of safety against tilting 11329 ÷ 6098 = 1.86.

Interlock Tension

Interlock tension is investigated for the full active pressure, including pressure due to surcharge. The maximum occurs at the dredge line.

Tension = PR = 1803 x 34.21 = 61,800 lb. per ft. = 5150 lb. per in.
For an ultimate strength of interlock of 16,000, factor of safety is 16000 ÷ 5150 = 3.1.

Cellular Docks in Clay

From the writer's experience in discussing cellular dock problems with engineers on many projects, the conclusion is drawn that there is great need for clarification of principles of design for cellular docks in clay. There have been a number of failures, and in practically every case the cause was failure to recognize that adequate horizontal shearing strength of the clay in the cells is the most important prerequisite for stability against failure by tilting. Cellular docks have been designed in very soft clay, sometimes overlain by mud, with the expectation that the cellular construction of itself would provide the necessary stability. In most cases stability was investigated by the "overturning" method, with the resultant passing through the "middle third", as previously discussed herein.

In assessing the suitability of a particular site for a cellular design, a simple rule for preliminary investigation is to compare the total active pressure with the sum of the horizontal shear value of the clay in the cells and the passive pressure on the outboard side, if any. The dock shown in Fig. 13 is a case in point. Consideration was first given to a cellular dock without predredging. But laboratory analysis indicated a unit shear for the very soft silt of only 250 psf. Total horizontal shear resistance of the silt in the cells

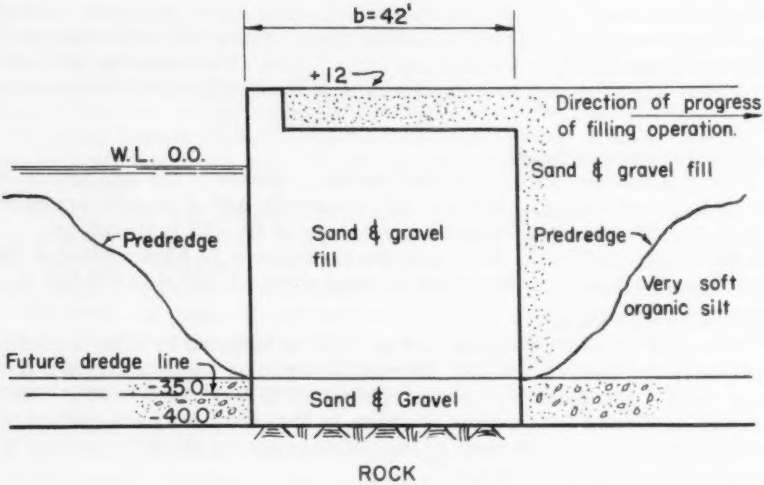


Fig. 13

would have been only a fraction of the active pressure, since the surcharge of the fill on the silt would have resulted in excessive lateral pressure. As indicated in Fig. 13, the silt was predredged and the cells filled with sand and gravel. Sand and gravel fill was placed immediately behind the cells with progress shoreward, in order to eliminate the possibility of a mud wave against the cells.

Definitions of symbols used in this paper are as follows:

γ = unit weight of soil
ϕ = angle of internal friction of sand
K_a = coefficient of pressure for sand
ρ_o = weight of overburden per sq. ft. plus surcharge if any
q_u = unconfined compressive strength of clay
s = shear value of clay = $\frac{1}{2}$ q_u
ρ_a = unit active pressure = ρ_o k_a for sand = ρ_o - 2s for clay
k_p = coefficient of passive pressure for sand
ρ_p = unit passive pressure = $\rho_o k_p$ for sand; and ρ_o + 2s for clay
P_a = total active pressure
w = unit weight of water
P = total active pressure causing interlock tension
f = coefficient of friction in interlock = 0.3
b = average width of cellular cofferdam or dock
R = radius of circle or arc in cellular cofferdam
h = height of cellular cofferdam

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MODEL STUDIES OF REMEDIAL WORKS FOR NIAGARA FALLS

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(Proc. Paper 1367)

SYNOPSIS

Model studies of the Niagara River and Falls were conducted to determine: (a) the effects of the additional diversions of the Niagara waters for power authorized under the U. S. - Canadian Treaty of 1950; and (b) the locations and designs of the remedial works required to compensate for these effects as necessary to preserve and enhance the beauty of the Niagara River and Falls.

Results of the model tests indicated that three separate remedial works would be required to ensure meeting the provisions of the 1950 Treaty for increasing diversions for power while preserving and enhancing the beauty of the River and Falls; (a) A Chippawa-Grass Island gated control structure extending out from the Canadian shore; (b) an excavation at the Canadian flank of the Horseshoe Falls, with a 100-ft crest fill at that flank; and (c) an excavation at the Goat Island flank of the Horseshoe Falls, with a 300-ft crest fill at that flank.

INTRODUCTION

The Niagara River carries the outflow from the four upper lakes of the Great Lakes system from Lake Erie to Lake Ontario. The vast storage capacity of the upper lakes results in a remarkably steady river discharge averaging about 200,000 cfs, although the discharge does vary somewhat daily, seasonally, and annually.

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The river has a total length of 36 miles, in which the total drop from the mean level of Lake Erie to that of Lake Ontario is 326 ft. At Niagara Falls the river has a vertical drop of about 160 ft, and an additional drop of about 140 ft occurs in the eight miles of cascades and rapids immediately above and below the Falls. The plan and profile of the river and Falls is shown on Fig. 1.

This concentration of fall within a relatively short distance, together with the large and steady discharge of the river, makes possible the economical development of hydro-electric power. Furthermore, the Falls of the Niagara and its natural surroundings constitute one of the most famous and spectacular scenic wonders of the world. Boundary-water treaty agreements between the United States and Canada regulate diversions of water from the river for power and reserve sufficient water in the river for scenic purposes.

The Prototype

Upper Niagara River

The Niagara River flows northward from Lake Erie near Buffalo Harbor through a funnel-shaped entrance much obstructed by shoals. For the first two miles it is little more than 1500 ft wide, with a maximum depth of 20 ft and velocities up to eight miles per hour. The river here is paralleled on the New York side by the Black Rock Canal, with a lift of about 5 ft, which permits passage of vessels between Buffalo Harbor and the Niagara River just below Squaw Island. Below this point the river is navigable through both channels around Grand Island, and a 21-ft channel has been dredged in the river from the Black Rock Canal to Tonawanda and a 12-ft channel from there to the docks at Conners Island, Niagara Falls, New York.

The four miles of river between the lower end of Grand Island and the head of the Cascades immediately above the Falls is known as the Chippawa-Grass Island Pool (see Fig. 2). At present, four intakes divert water from the Pool for power purposes: the Adams and Schoellkopf intakes near Grass Island on the United States side of the river, and the intakes for the Sir Adam Beck No. 1 and No. 2 plants (the latter just recently constructed) on the Canadian side near Chippawa. The intake for a plant to be built on the United States side of the river is to be located near Conners Island. The latter two plants are planned to utilize the increased diversions of water permissible under the treaty of 1950, which will be discussed later.

During the years 1942 to 1947, a submerged weir was built near the lower end of the Pool in the main channel leading to the Canadian side of the Falls. The purpose of the weir was to restore the Chippawa-Grass Island Pool to approximately the elevation from which it had been lowered by additional diversions for power authorized during the war period. The resultant raising of the Pool improved intake conditions, especially during ice runs, and increased the flow over the American Falls.

Niagara Cascades and Falls

Goat Island, situated at the brink of the Falls, divides the Niagara River into two channels, each half a mile long, one leading to the Canadian or Horseshoe Falls and the other to the American Falls. In each of these channels the water flows over ledges of limestone, scattered boulders, and broken

rock to form cascades and rapids with a total drop of 50 ft from the Chippawa-Grass Island Pool to the crest of the Falls. The uppermost ledge of rock acts as a natural weir at the head of the Cascades. Before construction of the submerged weir in the 1940's, only 5 per cent of the total flow over these ledges went down the channel leading to the American Falls, but this flow was nearly doubled by the weir, thus greatly improving the spectacle of these Falls.

The channel leading to the Horseshoe Falls is 3200 ft wide at the head of the Cascades and only 1200 ft wide, shore to shore, at the Falls. Portions of the Cascades are 6 to 12 ft in depth, but several small islands and numerous shoals have been formed by the irregular distribution of ledges and boulders. A large central shoal divides the flow in the Horseshoe Falls channel into two main channels, one near each shore, which converge toward the center part of the horseshoe. The intakes of the Ontario Power Company, the Toronto Power Company, and the Canadian Niagara Power Company plants are located along the Canadian shore of the Cascades.

At the Horseshoe Falls, there is a straight drop of 160 ft. Although the shore-to-shore width is only 1200 ft, the total length of the crest measured around the horseshoe is 2500 ft. Near both flanks, where the depth of water flowing over the crest was less than 1 ft before the recent improvement of the Falls, the normal color of the falling sheet of water was white in appearance and the sheet was broken at times because of its thinness. Toward the center of the horseshoe, the depth increases to a maximum of 12 ft. As the depth at the crest increases to 4 or 5 ft, a greenish color mingles with the white and greatly enhances the appearance of the Falls.

History of Power Development

Efforts to harness the Niagara waters for power development date back to the year 1725, when an uneconomical attempt was made in the Cascades above the Falls. Successful exploitation of the river for power began in 1877, followed by spasmodic and ever-increasing growth of power developments. Now, however, diversions of water for power are limited by boundary-water treaty agreements between the United States and Canada. The first such treaty, made in 1909 and 1910, permitted a permanent daily diversion from Niagara River above the Falls for power of 20,000 cfs by the United States and 36,000 cfs by Canada. Because of the increased need for power for defense activities, the two countries concluded agreements in 1940 and 1941 to utilize on a temporary basis an additional diversion of 26,500 cfs at Niagara Falls. In 1944 and 1948 the earlier agreements were modified to provide for small additional temporary diversions, and discussions that led to the treaty of 27 February 1950 were commenced.

The Problem, and Purpose of Model Studies

Treaty of 27 February 1950

On 27 February 1950, a new treaty was made between the United States and Canada concerning uses of the waters of the Niagara River. This treaty provides for remedial works to enhance the beauty of Niagara Falls and reserves sufficient water in the Niagara River for scenic purposes. In general, the treaty provides that a Falls flow of no less than 100,000 cfs will be maintained for scenic purposes during daylight hours throughout the tourist

season, and that at no other time will the flow over the Falls be less than 50,000 cfs. All water over and above these requirements and the requirements for domestic, sanitary, and navigation purposes are made available for power purposes, such water to be divided equally between the United States and Canada.

The treaty specifies that the two governments recognize "their primary obligation to preserve and enhance the scenic beauty of the Niagara Falls and River." The treaty further states that the two countries agree to complete remedial works found necessary to enhance the beauty of the Falls by distributing the waters so as to produce an unbroken crest line around the Falls for all flows.

Investigation Procedure

The International Joint Commission, United States and Canada, was requested by the two Governments to investigate and submit recommendations concerning the remedial works necessary to carry out the provisions of the 1950 treaty. The Commission then created the International Niagara Falls Engineering Board, drawn from the technical agencies of the two countries, and directed it to make the necessary investigation of the Niagara Falls and River and to report its findings and recommendations. The Board appointed a working committee consisting of representatives of agencies normally responsible for the types of work involved. The regular field organizations of the appropriate agencies were asked to perform the various types of surveys and studies needed, thus ensuring that the services of specialists available in both countries were utilized on various aspects of the problem as required.

Flow conditions in the Cascades and at the brink of the Falls was so complex in nature that remedial works to redistribute the flow over the crest of the falls were not susceptible of design by analytical procedures. This fact is pointed up by the 1928 report of the Special International Niagara Board, which recommended a step-by-step procedure of constructing a combination of submerged weirs and excavations with observation of the results of each step to guide the design of the next step.

In recent times, however, the hydraulic model has gained wide recognition as an invaluable tool for engineering design. Successful methods and techniques have been developed, and an impressive record of accomplishment has proven the reliability of conclusions drawn from model studies. Where applicable, such studies constitute a dependable method of solving complex hydraulic problems at a minimum expenditure of time and money.

The advantage of using this tool in the investigation of the Niagara remedial works was evident. Any given river flow, past, present or future, could be simulated at will. Remedial works in miniature could be inserted in the model and their performance studied under the full range of river conditions. It was necessary to determine future conditions in the river if no remedial works were constructed, and the locations and designs of the remedial works required to correct these conditions as necessary to preserve and enhance the beauty of the Falls. The Engineering Board was of the opinion that only by model tests could reliable information of this nature be obtained.

Accordingly, the major phase of the engineering studies necessary for design of the remedial works was accomplished by means of two hydraulic model studies. One model study was conducted by the Hydro-Electric Power

Commission of Ontario at Islington near Toronto, while the other was conducted by the U. S. Army Engineer Waterways Experiment Station at Vicksburg, Mississippi.

There were several reasons for utilizing two model studies. The preservation of Niagara Falls being an international matter, it was necessary that both countries be satisfied as to the validity of the solution proposed. Also, in view of the far-reaching importance of the matters at issue and the complexity of the problem, it was considered desirable to utilize fully the available technical forces of both countries. The use of two models, similar but not exactly the same in scale and complementary in coverage, would make possible a constant check on test findings and lend assurance to the reliability and accuracy of the results obtained.

Purpose of the Model Studies

The general purpose of the model studies was to work out the detailed procedures for implementing the provisions of the Treaty of 27 February 1950. The achievement of this broad purpose involved: (a) establishing the conditions that would exist in Niagara River and Falls under the future increased diversions for power permitted under the Treaty of 1950 if no remedial works were constructed; and (b) determining the locations and designs of the remedial works that would be required to correct these conditions as necessary to preserve and enhance the beauty of the Falls.

The Models

Canadian Model

The Islington Model, constructed by the Hydro-Electric Power Commission of Ontario, reproduced the Niagara River from the lower end of Grand Island to the Rainbow Bridge below the Falls. This model, constructed to linear scale ratios (Model: Prototype) of 1:250 horizontally and 1:50 vertically, was 95 ft long and 37 ft wide. A general view of this model is shown on Fig. 3.

United States Model

The remainder of this paper is concerned only with the Niagara River and Falls model constructed by the U. S. Army Engineer Waterways Experiment Station at Vicksburg. This model reproduced about 26 miles of the Niagara River, extending from 11,500 ft above the Peace Bridge at Buffalo to the Rainbow Bridge which is located about 5000 ft below the Falls (see Fig. 4 for location map). The upper limit of the model extended far enough into Lake Erie to provide accurate reproduction of flow entering the Niagara River from the lake, while the lower limit included the gorge below the Falls for pictorial purposes. Between these extremities were reproduced the Falls and Cascades, the existing and proposed power intakes, Goat Island, Grand Island, and other significant topographical features. The reason for extending the model up to include a portion of Lake Erie was to permit studies of the extents and timings of the effects in the upper river and lake of remedial and regulating works in the vicinity of the Cascades and Falls.

The Niagara River and Falls model was constructed to linear scale ratios of 1:360 horizontally and 1:60 vertically, giving a model 260 ft long with a maximum width of 125 ft. Selection of these scale ratios was based on the following considerations:

a. Linear scale distortion (use of different scales for horizontal and vertical dimensions) was necessary because the extremely shallow depths in the most critical section of the prototype required a vertical scale large enough that its application to the great horizontal dimensions of the prototype would have produced a model of impractical size and cost.

b. Known physical and hydraulic characteristics of the Niagara River indicated that these linear scales would permit reproduction of the proper roughness factors and hydraulics of the prototype without appreciably altering the basic shapes and flow characteristics of the model channels.

c. Previous experience with similar problems indicated that such a model would provide satisfactory solutions of the problems involved.

The model was of the fixed-bed type, with all channel and overbank areas molded in concrete. The concrete formed a thin shell about 2 inches thick, and in critical areas removable concrete sections were molded so that the existing channel conditions and various proposed improvement plans could readily be changed to represent any desired condition. A general view of this model is shown on Fig. 5.

Field Data for Model Construction

Field surveys used to reproduce in the model the river bed and shore-line topography of the low-velocity reaches of the river were obtained by conventional survey methods. However, the section of Niagara River extending from the crest of the falls upstream about 4000 ft, which section was of paramount importance to the model study, presented a very difficult and unusual surveying problem. Few usable data on this reach were available, and the area is rendered entirely inaccessible by the strong currents and violent turbulence of the Cascades. However, by ingenious methods including soundings from helicopters and balloons, together with extra precautions with normal surveying equipment, thorough surveys were made of the area for model construction purposes.

Model Appurtenances

The bridges and the existing and proposed power intakes along the river were precisely reproduced in the model. The intakes were constructed of wood, and the bridges with wood piers and sheet metal trusses. Flow into the intakes was regulated by standard gate valves and measured by Van Leer Weirs. Provisions were also made for measurement of flows in the channels around Grand Island and flows over the American and Horseshoe Falls. Water-surface elevations in special problem areas were measured by means of portable point gages. During the course of the tests it was found desirable to measure the flow over the Horseshoe Falls in 100-ft increments around the crest, and this was accomplished by means of a specially constructed scoop that diverted the flow over any 100-ft section through one of the Van Leer Weirs for measurement.

Hydraulic Adjustment of Model

The first step in the model testing program was the adjustment of channel roughness so that the model would accurately reproduce in detail all of the

observed hydraulic phenomena of the prototype. This procedure fell naturally into two separate operations: first, adjustment of the relatively low-velocity channel upstream from the Cascades, including the obtaining of the proper distribution of flow around Grand and Goat Islands; and second, adjustment of the high-velocity, turbulent Cascades and Falls section.

The section of river from Lake Erie to the Cascades was adjusted until the water-surface elevations at 18 gages agreed with corresponding gage readings obtained in the river for this special purpose with a discharge of 223,488 cfs. Water-surface elevations in the model at all standard gages were further checked against prototype elevations computed from a gage-relations formula for flows ranging from 150,000 to 240,000 cfs to ensure that the model was accurately adjusted for the entire range of discharges that would be used later in the testing program.

Adjustment of the Cascades section of the model was a tedious cut-and-try process. The prototype data used for this purpose consisted of: (a) a water-surface contour map, known as the "searchlight survey," obtained by determining locations and elevations by means of a reflected beam of light moved at random over the Cascades at night; (b) a vertical aerial photograph of the Cascades and Falls taken during a large ice flow and depicting general streamlines through the Cascades area; (c) a float survey indicating the general distribution of flow around the crest of the Falls; and (d) water-surface elevations at 23 gages located in the Cascades along the Canadian and Goat Island shore lines.

In the reach above the Cascades, proper degrees of roughness were obtained with wire screen fitted to the bed of the model. In the Cascades area, the roughness required to reproduce the turbulence, streamlines, and water-surface elevations was obtained in the model by means of stucco, small embedded rectangles of sheet metal, and small rocks. These mechanical additions, while tending to mar the appearance of the model, were necessary for technical hydraulic reasons.

Model Tests Without Remedial Works

The first series of tests after adjustment of the model was made to determine the conditions that would be brought about in Niagara River and Falls by the future increased diversions for power permitted under the Treaty of 1950 with no compensating remedial works provided. The total diversion permitted by the Treaty includes all flows in excess of those required to maintain total Falls flows of 100,000 cfs during daylight hours of the tourist season and 50,000 cfs at all other times. The model tests of these conditions indicated that without construction of additional remedial works the following conditions would result from the increased permissible diversions:

a. The Chippawa-Grass Island Pool level would be drawn down as much as four feet below its normal level, thereby exposing considerable areas of the river bed previously covered, particularly in the vicinity of the head of Goat Island. The drop would vary from zero to three ft during daylight hours of the tourist season and from two to four ft at other times, depending upon river discharge. Lowering of the Pool would result in some lowering of levels of Lake Erie.

b. Because of the lowering of the Pool level, flow over the American Falls

would drop well below that required for a satisfactory scenic spectacle. The present satisfactory American Falls flow of 11,500 cfs for an average river discharge of 200,000 cfs would be reduced by the maximum permissible diversion to only 4600 cfs during the tourist season days and 2500 cfs at other times.

c. Under maximum permissible diversion for average river discharge the Horseshoe Falls would have a flow of only 95,000 cfs during tourist season days, producing unsatisfactory flows at the flanks. During other times the Horseshoe Falls flow would be only 47,000 cfs, which would leave both flanks dry. Even under conditions existing before the authorized increase in diversions, with an average flow of 105,000 cfs over the Horseshoe Falls, the flanks were inadequately covered.

d. About 12 hours time would be required to change the total Falls flow from 50,000 to 100,000 cfs, or vice versa, owing to the slow response of the Chippawa-Grass Island Pool level to changes in diversions. Therefore, only a small part of the extra diversion authorized at night during the tourist season could be used, as it would be necessary to start reducing diversions far in advance in order to build up the Pool to the level required for a flow of 100,000 cfs over the Falls by 8:00 a.m. Furthermore, the resulting lowering of the Pool would adversely affect the output of power plants drawing water from the Pool.

Performance Criteria for Remedial Works

In view of the intolerable conditions that would result from the authorized increased diversions if no remedial works were constructed, it was considered imperative that remedial works be provided to improve the distribution of flow over the crest of the Horseshoe Falls, to maintain the satisfactory flow conditions at the American Falls, and to control the levels of the Chippawa-Grass Island Pool. The maintenance of the existing relationship between river flow and Pool level was considered essential to preserve the existing conditions and appearance of the Niagara River upstream from the Pool and to ensure that Lake Erie levels and corresponding outflows would remain unaffected, thus protecting interests upstream which otherwise might be affected adversely by general lowering or rapid fluctuation of the Pool level.

Accordingly, the International Niagara Falls Engineering Board requested that model tests be made to determine the locations and designs of remedial works that would ensure meeting the following criteria:

a. A dependable flow of water over the American Falls and Rapids approximating the satisfactory intensity experienced under existing conditions.

b. A dependable and ample flow of water over both flanks of the Horseshoe Falls to provide an unbroken crestline, the intensity of flank flows to be such as to satisfy the following requirements:

1) For a total flow over the Falls of 100,000 cfs, a flow per foot of crest length of 6 to 8 cfs over the Goat Island flank and 10 to 12 cfs over the Canadian Flank.

2) For a total flow over the Falls of 50,000 cfs, an unbroken curtain of flow from shore to shore.

c. Maintenance of the existing relationship between the total river flow and the level of the Chippawa-Grass Island Pool.

d. Ability to meet promptly the changes in permissible power diversions while assuring flows of either 50,000 or 100,000 cfs over the Falls.

The remedial works for meeting the above criteria fall naturally into two categories: first, works to preserve the existing range of levels in the Chippawa-Grass Island Pool and maintain an adequate flow over the American Falls; and second, works to improve the distribution of flow along the crest of the Horseshoe Falls. Works of the first category would be located in the Pool just upstream from the head of the Cascades, and those of the second category in the Cascades upstream from the Horseshoe Falls. In the model tests of remedial works, the general procedure was to study first the control structure in the Chippawa-Grass Island Pool and then the remedial works above the Horseshoe Falls, because the design and location of the upper works would have some effect on the lower ones.

Model Tests of Control Structure in Pool

Tests carried out with a gated-dam control structure at several locations at and downstream from the exiting submerged weir near the head of the Cascades indicated that the performance at all of these locations was essentially the same. From the standpoint of feasibility and economy of construction, it was decided that the dam should be located parallel to and about 250 ft downstream from the submerged weir, as shown on Fig. 2. The tests indicated that the structure should start from the Canadian shore because the deep channel which must be intercepted to provide efficient control lies near the shore. This deeper channel also offers less likelihood of ice grounding in the channel in the vicinity of the dam.

Extensive tests were conducted to determine the optimum length of the control structure. Structures extending from the Canadian shore for various lengths were tested, including one extending across the entire river. In all these tests the entire control structure consisted of piers with 100-ft gated openings between them. In order to minimize interference with the free passage of ice, the gate sills were placed at an elevation not exceeding that of the river bed, or that of the submerged weir for the portion of the dam opposite the weir. Also, in certain of the tests in which the dam extended only partly across the river, experiments were made with an added short structure extending from the United States shore into the channel leading to the American Falls.

In general the results of these tests were as follows:

a. For the main gated control structure to be built out from the Canadian Shore, a minimum length of 1705 ft would be necessary to regulate the Pool under future conditions of diversion to the same levels that previously existed for the same river flows. This structure would also maintain satisfactory flow over the American Falls, and would permit changing of the total flow over the falls from 100,000 to 50,000 cfs and vice versa without a change in pool level. Thus, the 1705-ft structure satisfied all of the established performance criteria.

b. A 450-ft-long gated structure near the United States shore, although not

necessary for Pool control, was found to be of some value in controlling flow into the channel leading to the American Falls, especially at high river flows. However, this feature was dropped from further consideration because its cost would be out of proportion to the resulting benefits.

As stated above, the model tests indicated that the strict maintenance of the existing relationship between river discharge and Pool level, allowing for no tolerance at any discharge, would require a minimum length of control structure of 1705 ft. From the practical standpoint, however, the plan of operation of the control structure would logically allow a reasonable tolerance in the daily and monthly average deviation of the Pool from its normal level. Furthermore, some small deficiency in pool level at high discharges would probably be beneficial in reducing flood damages upstream. Accordingly, it was decided by the International Niagara Falls Engineering Board, on the basis of a balancing of all considerations, that the control structure would be constructed to a length of 1550 ft.

Model Tests of Remedial Works in Cascades

In the process of developing successful remedial works for improvement of flow conditions at both flanks of the Horseshoe Falls, many different schemes were tested in the model Cascades above the Falls. Some of these schemes involved weirs in the deeper channels to intercept and divert flow to the flanks of the Falls; others involved excavations on the flanks extending far enough upstream to divert flow to the flanks; and some consisted of both weirs and excavations. Consideration was also given to moderate shortening of both flanks by fills, not only to help intensify flow at the flanks and reduce excavation, but also to provide excellent vantage points from which the Falls and Cascades could be viewed at close range.

Tests of flank excavations were begun with small quantities of excavation, which were progressively increased and tested until the criteria for flow over the flanks were met. Each excavation scheme was tested both with and without the viewing fills at both ends of the crest. These tests culminated in the development of a satisfactory plan consisting of the following items, as shown on Fig. 2.

a. Flank Excavations. Excavation of an estimated 64,000 cu yd of rock on the Canadian flank and 24,000 cu yd on the Goat Island flank to deepen these areas and tap the deeper waters upstream, so as to divert sufficient water to provide satisfactory flows over both flanks.

b. Flank Fills. Construction on the Canadian flank of a crest fill 100 ft wide and extending about 100 ft upstream, and on the Goat Island flank of a crest fill about 300 ft wide and extending about 300 ft upstream. Both fills would be surrounded by concrete retaining walls faced with stone to blend into the natural surroundings, and both would be placed to the grades of the adjacent improved park areas and properly landscaped, thus providing excellent vantage points for viewing the Cascades and Falls.

With this scheme installed in the model the intensities of flow established as criteria during a total Falls flow of 100,000 cfs were exceeded on both flanks, and there was a complete curtain of flow over the Falls from shore to shore during a total Falls flow of 50,000 cfs. Figure 6 shows a comparison of flow conditions over the model Horseshoe Falls under natural unimproved conditions and with the remedial works installed.

Alternate schemes involving weirs, with and without flank excavations, were also developed which gave the required crest flows. However, the required weirs were so high as to give the Cascades an artificial appearance. Furthermore, their construction and maintenance would be difficult and hazardous, and their cost no less than that of the excavation plans. Consequently, the schemes involving weirs were rejected.

SUMMARY

In summary, the proposed plan of remedial works developed by the model tests consists of three separate works, which are considered necessary to ensure that the provisions of the 1950 Treaty will be fully met:

a. A Chippawa-Grass Island Pool gated control structure extending 1705 ft out from the Canadian shore.

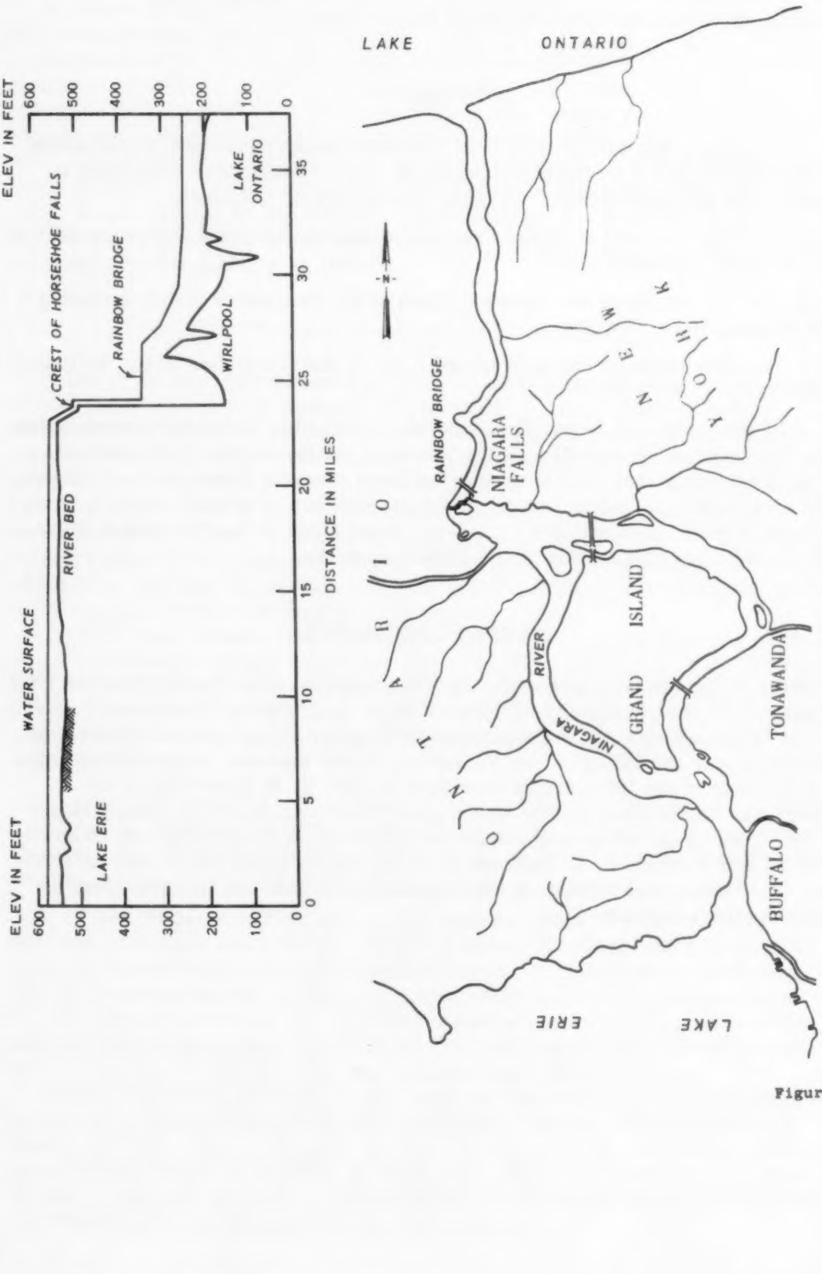
b. An excavation at the Canadian flank of the Horseshoe Falls, including a 100-ft crest fill at that flank.

c. An excavation at the Goat Island flank of the Horseshoe Falls, including a 300-ft crest fill at that flank.

A Chippawa-Grass Island Pool control structure is now under construction. The two excavations and fills at the flanks of the Horseshoe Falls have already been completed, and the resulting flows over the flanks of the falls provide an excellent confirmation of model predictions. Figure 7 shows a comparison of flow conditions over the Goat Island flank of the Horseshoe Falls before and after installation of the remedial works.

ACKNOWLEDGMENTS

Much of the material presented in this paper has been drawn from the 1953 Report of the International Joint Commission of the United States and Canada on the "Preservation and Enhancement of Niagara Falls," which contains the results of both the Islington and Vicksburg model studies. Acknowledgment is also made of the information furnished by Mr. E. B. Lipscomb of the Waterways Experiment Station who directly supervised the Vicksburg Model study. This model study was conducted for the U. S. Army Engineer District, Buffalo, and Messrs. S. B. Hunt and J. G. Weinrub of that office took an active part in obtaining and furnishing the required field data and in developing the model testing program.



NIAGARA RIVER PLAN AND PROFILE

Figure 1

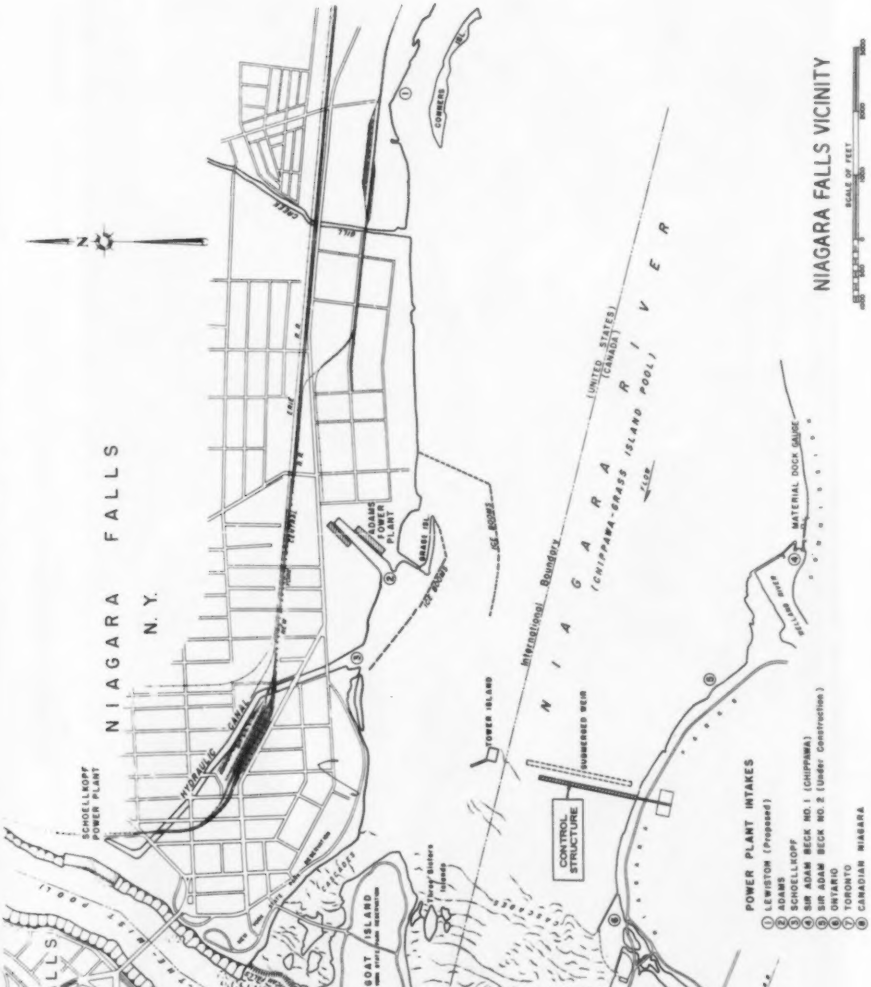

NIAGARA FALLS VICINITY
SCALE OF FEET
NIAGARA FALLS N.Y.
NIAGARA FALLS ONT.
NIAGARA RIVER
(CHIPPAWA-GRASS ISLAND POOL)
International Boundary
(UNITED STATES)
(CANADA)
FLOW
HYDRAULIC CANAL
SCHOELLKOPF POWER PLANT
ADAMS POWER PLANT
GRASS ISL.
ICE BOOMS
TOWER ISLAND
SUBMERGED WEIR
CONTROL STRUCTURE
GOAT ISLAND
Three Sisters Islands
FILL
EXCAVATION
ONTARIO POWER PLANT
CANADIAN NIAGARA POWER PLANT
TORONTO POWER PLANT
MATERIAL DOCK GAUGE
WELLAND RIVER
CAYUGA ISL.
POWER PLANT INTAKES
1 LEWISTON (Proposed)
2 ADAMS
3 SCHOELLKOPF
4 SIR ADAM BECK NO. 1 (CHIPPAWA)
5 SIR ADAM BECK NO. 2 (Under Construction)
6 ONTARIO
7 TORONTO
8 CANADIAN NIAGARA

Figure 2

NIAGARA RIVER AND FALLS MODEL (ISLINGTON)

Figure 3

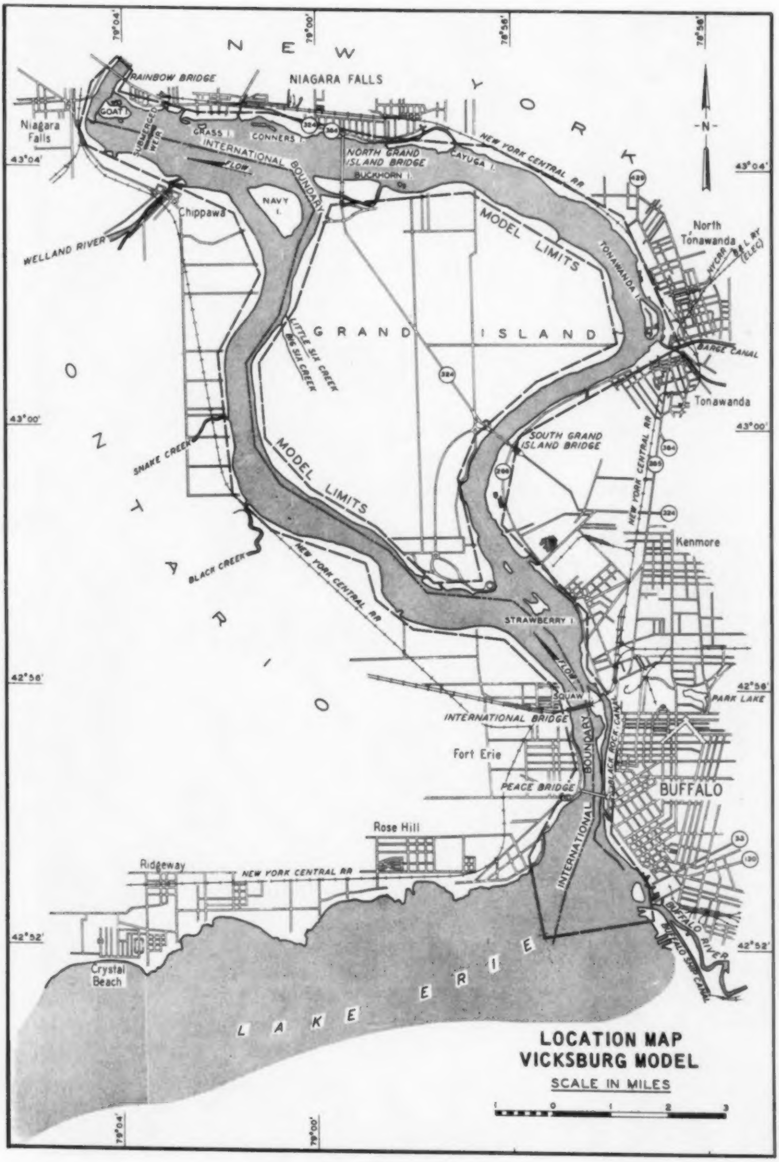
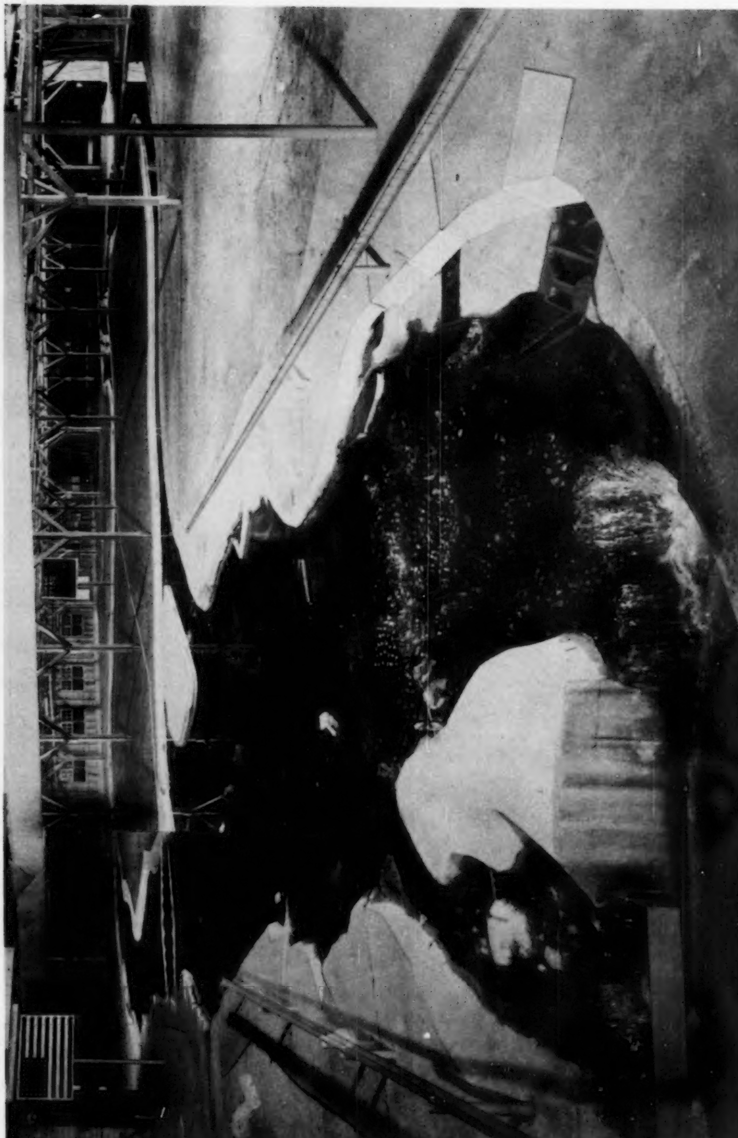
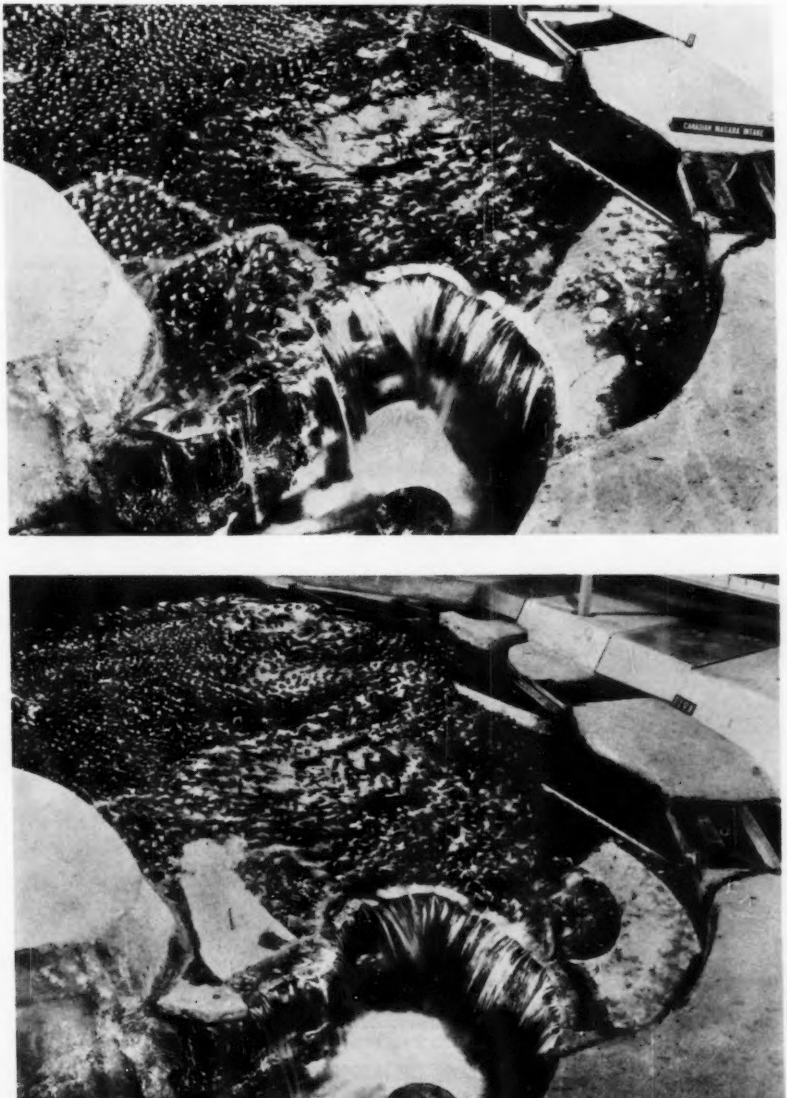

N E W Y O R K
NIAGARA FALLS
RAINBOW BRIDGE
Niagara Falls
GOAT I.
SUBMERGED WEIR
GRASS I.
CONNERS I.
INTERNATIONAL BOUNDARY
FLOW
NORTH GRAND ISLAND BRIDGE
CAYUGA I.
BUCKHORN I.
NEW YORK CENTRAL RR
NAVY I.
Chippawa
WELLAND RIVER
MODEL LIMITS
TONAWANDA I.
North Tonawanda
GRAND ISLAND
LITTLE SIX CREEK
BIG SIX CREEK
BARGE CANAL
Tonawanda
O N T A R I O
SNAKE CREEK
SOUTH GRAND ISLAND BRIDGE
BLACK CREEK
Kenmore
STRAWBERRY I.
SQUAW
PARK LAKE
INTERNATIONAL BRIDGE
Fort Erie
BLACK ROCK CANAL
PEACE BRIDGE
BUFFALO
Rose Hill
Ridgeway
BUFFALO RIVER
Crystal Beach
L A K E E R I E
LOCATION MAP
VICKSBURG MODEL
SCALE IN MILES
43°04'
43°00'
42°56'
42°52'
79°04'
79°00'
78°56'

Figure 4



NIAGARA RIVER AND FALLS MODEL (VICKSBURG)

Figure 5



HORSESHOE FALLS (MODEL)
ABOVE: NATURAL CONDITIONS BELOW: REMEDIAL WORKS INSTALLED

Figure 6

GOAT ISLAND FLANK OF HORSESHOE FALLS
ABOVE: NATURAL CONDITIONS MAY 1954 BELOW: REMEDIAL WORKS INSTALLED DECEMBER 1954

Figure 7

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

OFFSHORE BREAKWATERS

Richard Silvester[1]
(Proc. Paper 1368)

SYNOPSIS

The determination of wave height and wave pattern for waves diffracted by an infinitely long breakwater, or by a breakwater gap, has been based on the theoretical solution for optical diffraction. Correct results are obtainable at distances in the shadow zone many wave lengths from the breakwater, but within an area of three wave lengths results are erroneous. Offshore breakwaters of limited length, although basically the same as the infinitely long breakwater, have peculiarities which must be studied if their operation, in preventing beach erosion for example, is to be successful. The main factor to be considered is wave pattern close to the breakwater. Model tests for wave patterns with two different lengths of breakwater and waves of different steepnesses are described and results presented in a form suitable for application. It is hoped that this article will stimulate research into this and other aspects of offshore breakwater operation so that engineers can be supplied with sufficient data for design purposes.

SYMBOLS

Symbols used in order of appearance in the text.

H_T (ft.)	height of wave component of period T (see Ref. 7).
T (sec.)	period of wave.
H (ft.)	height of wave.
L_0 (ft.)	deep water wave length.
d (ft.)	still water depth.
E (lb.)	energy per ft. width of crest per wave.
w (lb/ft^3)	unit weight of sea water.

Note: Discussion open until February 1, 1958. Paper 1368 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 83, No. WW 3, September, 1957.

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INTRODUCTION

Breakwaters constructed parallel to and some distance from the shore have not come into general use as a means of preventing beach erosion, even though their effectiveness in slowing down, if not completely stopping, littoral movement has been amply illustrated as long ago as 1930[1].

One reason for this lack of support for the offshore breakwater is its greater cost over the shore bound structure, not only in money but possibly in human life also; for their construction in the open sea with present day equipment is, to say the least, hazardous.

For this lack of support must also be attributed the lack of information on the action of offshore breakwaters. Unlike the land based groyne which all stop littoral drift immediately on installation, sometimes effectively other times not so effectively, the offshore structure appears to be somewhat unpredictable. Even a publication such as that of Reference 2 can only advise the construction of a short breakwater initially and, after observing its action, add to it until desired results are obtained. It is obvious that such an empirical approach does not suit persons responsible for preparing estimates and raising the necessary funds.

Mathematical Solution

Although the mathematical solution for the height of diffracted waves behind an infinitely long breakwater has been applied from those of optical, sound and electromagnetic wave diffraction[2,3,4,5,6], it has been realised always that "only model tests will make it possible to state our assumptions will lead to errors too large for practice"[4].

Such experimental verification has not met with extraordinary success. For example, to quote Carr and Stelzriede (1952)[3] "The agreement between experiment and theory, while not exact, is reasonably close, and supports the important general conclusions of the theory." Putnam and Arthur (1948)[6] remarked "Although the accuracy with which the diffraction coefficients could be determined experimentally is not all that might be desired, it is possible to make the following conclusions etc."

When looking for a reason for this apparent inaccuracy in the theoretical solution it must be borne in mind that it is founded upon the sinusoidal wave form. Any deviation away from this wave form must affect not only the distribution of wave height but also the paths followed by the wave crests. Wave steepness may be the factor causing such deviation. This in turn is dependent upon wave height, wave period and depth of water.

Wave Steepness

The waves used in the experimental studies mentioned above were generally steep in nature. Putman and Arthur (1948)[6] used deepwater waves of steepness = 0.035. Carry and Chapus (1951)[5] used waves in water whose depth was about $\frac{1}{8}$th that of the wave length, wave heights were not included in the report. Carr and Stelzriede (1952)[3] did not record data on their experimental waves, pointing to the fact that it was considered unimportant in the ultimate results.

When studying littoral drift average wave conditions are more important than maximum or minimum, thus average wave steepness should be used in tests rather than maximum values. To do this maximum steepness of ocean waves must first be defined.

If the hindcasting method of Darbyshire is to be accepted[7] than maximum values of $H_T/_T = 0.44$ can be expected for the component waves in the wave spectrum. Although by interference of wave groups larger and smaller waves will occur this value should be representative of average conditions.

Wave steepness in deep water $\frac{H}{L_0}$ then becomes:

$$\frac{H}{5.12T^2} = \frac{0.44}{5.12T} = \frac{0.086}{T}$$

Changes in wave height and wave length can be calculated for waves travelling through shoaling water[2] so that steepness at various values of $\frac{d}{L_0}$ or $\frac{d}{L}$ can be determined (see Figure 1). It can be seen that a steepness of 0.035 in deep water can only represent waves of 2.5 seconds or less whereas for shallower conditions (eg. $\frac{d}{L_0} = 0.01$) it could represent almost a 15 second wave. The shape of the wave in each case would be quite different, the former being closer to the sinusoidal form.

Wave Pattern

Offshore breakwaters may be built to provide calm water for vessel anchorage in which case height of diffracted waves assumes importance, but mainly their usefulness would be in the sphere of erosion mitigation. In this case the pattern of the waves behind the breakwater is paramount in the formation or non-formation of a tombolo, especially within one or two wave lengths of the breakwater. It is generally accepted that within this region the theoretical solution is somewhat in error due mainly to the fact that until attenuated by the diffraction the waves may not be in true sinusoidal form. As the waves progress further into the shadow zone they follow a pattern and a height distribution well represented by the mathematical solution of optical diffraction.

It was decided to check whether factors such as wave steepness, caused by differing the wave height, wave length or water depth, had any effect on the wave pattern behind a short offshore breakwater. Measurements would be compared to the theoretical solution.

Experimental Procedure

A model basin was available with a wave machine capable of producing waves of various heights and periods. Waves to a scale of 1/60 of the prototype were produced to a natural scale by using the time scale of $1/\sqrt{60}$. Since the waves were generated in shallow water the blade of the machine not only rotated about an axis at the bottom of its face but the base also oscillated.

Two sections of the model were partitioned off in front of the blade, one with a depth equivalent to 5.75 fathoms (0.575 ft.) and the other with a depth

of 3 fathoms. In the former the concrete floor of the model was the base, whilst in the latter $\frac{1}{8}$" metal screenings were used to build up to the level. Frictional and percolation losses due to the screenings were negligible as judged by measured values of L and theoretical values.

The equivalent lengths of the breakwaters tested were 240 and 360 feet respectively. The characteristics of the waves used are listed in Table 1. It is seen that the two periods of 7 seconds and 9 seconds were reproduced fairly closely. The crest lag on the centre line of the breakwater was traced by placing pegs along this line at 2 foot intervals (equivalent to 120 feet) and locating others on a parallel line running through one end at the point of the crest when the same crest was at the centre peg. Height of the diffracted waves was not measured.

Experimental Results

In the 5.75 fathom depth it was difficult to trace wave crests for a great distance behind the breakwater due to interference of waves from each side and reflections from the partitions. Only two traces of the crest were obtainable. With the 3 fathom depth the waves were steeper and could be traced more easily.

It was found that although the crest lag differed a little for the various wave steepnesses, due probably to the slightly varying periods, no trend was visible and the differences were put down to experimental error. In Figure 2, therefore, where the wave patterns are recorded, each curve is representative of three or four waves of different steepness (see Table 1). The theoretical solutions[2] are recorded in Figure 2 by thin lines.

Crest lag is not affected by wave period as seen in Figure 2 where traces for 7 and 9 second waves are almost identical, even within two wave lengths of the breakwater. Also, comparison of wave patterns in the different depths of water shows that wave length does not materially affect crest lag.

Crest lag is a maximum at the moment when the diffracted wave reaches the centre line of the breakwater. As soon as the waves from either side join forces the crest at the centre accelerates and decreases the lag to a point about 600 feet behind the breakwater after which the decrease is slow.

Figure 3 has been prepared showing $\frac{\text{crest lag}}{\text{breakwater length}}$ versus distance of crest behind the breakwater (on centre line) both for the model tests and for the theoretical solution of the infinitely long breakwater. It can be seen how the latter gives the correct result beyond 600 ft. from the breakwater. It would appear that the longer the offshore breakwater the closer is the mathematical solution. Whether model tests for an infinitely long breakwater would actually be predicted by it at points close to the breakwater requires to be checked.

Comparison of wave patterns for the two lengths of breakwater can be made from Figure 4 where the tips of the breakwaters have been made coincidental. Here as in Figure 3 it can be seen that the wave pattern for the longer breakwater is nearer to the mathematical solution for the infinitely long breakwater. It is seen that until the diffracted waves meet at the centre they follow a well defined pattern for either length of breakwater.

Discussion

For an offshore breakwater to form a tombolo and intercept littoral drift or for it to cause a protruberance of the coast without completely blocking the passage between the structure and the shore, certain conditions are necessary. Assuming constant depth of water these conditions include wave pattern and wave height—wave pattern because the equilibrium shape of the beach depends upon this and wave height because it affects the steepness which in turn affects the type of action the wave has on the beach. Of course, the contours of the ocean bed will materially affect the processes taking place.

A breakwater can be constructed far enough offshore for the crest lag at the shore to be almost zero such that the shore line will not be changed at all. This is axiomatic.

Again, a breakwater can be constructed at such a distance offshore as to have a crest lag sufficient to cause a protruberance on the coast. This is caused by a littoral current formed by the breaking of the waves at a slight angle to the shore. If the approaching waves are parallel to the breakwater and the shore (see Figure 5) when there is no other littoral current present erosion will take place at points either side of the centre line of the structure in order to provide material at the centre.

In fact the shore will tend to build further out than the wave pattern would indicate because the littoral currents set up carry sediment out into the deep water not only in suspension but by saltation action also.

The importance of saltation in littoral drift is only just being realised(8). When shallow water waves causing oscillatory horizontal movement of water particles near the ocean floor, throw sediment into suspension momentarily, this sediment moves in any direction that a slight current may take it. Ripples form on the bed parallel to the crests of the waves and have dimensions dictated by several factors(9), the main ones being the wave height and the wave period. In order to travel the particles jump from ripple to ripple or along ripples depending upon the direction of the current, which does not have to be of great velocity to overshadow any mass transport effect(9).

In the situation as sketched in Figure 5 the wave height is greatest along the centre line of the breakwater (by observation) and this would make for greater saltation action there, making it easier for the outmoving currents to transport sediment.

When there is already a littoral drift along the shore due to waves approaching at some angle to it the presence of an offshore breakwater (see Figure 6) would cause greater erosion on the "leeward" side and silting on the "windward" side. The protrusion of the shore (if great enough) would cause a venturi effect behind the breakwater and would expedite the passage of material through the gap.

It has been shown in the tests described above that for constant depths the pattern of diffracted waves around offshore breakwaters is independent of period, height or the resulting steepness. It is also independent of the depth. But such ideal conditions do not exist and the slope of the ocean bed influences waves by the process of refraction. Thus waves will tend initially to straighten out more quickly than shown by Figure 3. As wave velocity influences wave refraction then wave period and water depth affect the wave pattern. As the protruberance on the coast forms the contours will blend to this natural pattern of the waves.

Wave Energy

Diffraction of ocean waves is defined as the phenomenon by which energy is transmitted laterally along a wave crest(2). Energy is expressed fairly accurately by

$$E = \frac{1}{8} w L H^2$$

Assuming no loss of energy as a wave approaches the shore the steepening that takes place in the shoaling water causes a greater proportion of this energy to be contained in the H term than the L term. Thus the energy in essence changes from kinetic to potential.

In a graph of H and L iso-energy curves can be drawn (see Figure 7) as can values of steepness $\frac{H}{L}$ and proportions $\frac{H^2}{L}$ of energy. It is seen immediately that increase in steepness implies increase in the proportion $\frac{H^2}{L}$. It is reasonable to suspect that the action of energy transmission along a wave crest would differ for various degrees of steepness of the wave, especially within one or two wave lengths of the breakwater.

Within this region also the effect of varying degrees of roughness of the breakwater needs to be studied. To date all experiments have been conducted with good reflective walls at the rear of the breakwaters whereas, in practice, they are extremely rough and tend to dissipate the energy of waves in contact with them. In the transmission of energy along a wave crest a gradient must be present for it to occur at all and if this gradient is maintained by such dissipation the wave height distribution must differ from that existing when energy is retained or maybe even reflected on a smooth back face. Study of diffraction of sound waves within one or two wave lengths of the barrier has shown the absorbant qualities of the barrier to be important. Of course, beyond about three wave lengths in the shadow zone the spreading of the wave becomes paramount in energy distribution and end conditions are not effective at all.

Both these fields require urgent enquiry if useful information about offshore breakwaters is to be supplied to engineers.

CONCLUSIONS

From the limited tests on wave patterns described herein the following conclusions may be drawn.

(1) The pattern of waves behind an offshore breakwater in an area of ocean with a horizontal bed is not affected by wave steepness and thus is not affected by wave height, wave length, wave period nor still water depth.

(2) Crest lag behind an offshore breakwater is dependent upon the length of the breakwater and decreases rapidly from a maximum of 65% of this length just behind the breakwater to 11% within 700 feet of the breakwater.

(3) The mathematical solution for crest lag behind an infinitely long breakwater gives correct results in the shadow zone beyond 600 feet of the breakwater, but gives values that are too small within 600 feet.

(4) The mathematical solution for crest lag is more correct the longer the offshore breakwater within the limitations of (2) above.

(5) If the factor of sloping bottom can be accounted for, conclusion (1) leads to the result that study of the effect of offshore breakwaters on littoral drift can be conducted in distorted models.

Discussion of the nature of ocean wave diffraction and of past experimental verification leads to the conclusion that:

(6) There is a great need for model tests on the wave pattern and height distribution behind breakwaters of limited length with waves of different degrees of steepness and breakwaters of different degrees of roughness on the shadow zone face. Especially should the phenomena be studied within the region up to three wave lengths from the breakwater.

ACKNOWLEDGMENTS

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TABLE I

Still water depth[a]	3 fathoms							5.75 fathoms						
Wave Period (secs.)	7.2	7.1	7.1	7.0	9.2	8.9	8.5	7.2	7.1	7.1	7.0	9.2	8.9	8.5
Wave Height (ft.) (b)	5.64	4.02	3.18	2.46	8.70	6.30	4.80	5.58	3.88	2.76	2.34	7.62	4.86	3.12
Wave Length (ft.) (c)	155	160	165	170	205	205	210	230	215	210	205	285	280	265
Wave Length (ft.) (d)	162	159	159	156	212	205	194	207	204	204	200	281	270	256
Height/Length (e)	.036	.025	.019	.015	.042	.031	.023	.025	.018	.013	.011	.027	.017	.012
Depth/Length (e)	.116	.112	.109	.106	.088	.088	.085	.15	.161	.164	.168	.121	.123	.13

(a) All measurements in terms of prototype.

(b) Measured in line with breakwater.

(c) As measured in model.

(d) As calculated - reference 2.

(e) Length as measured in model.

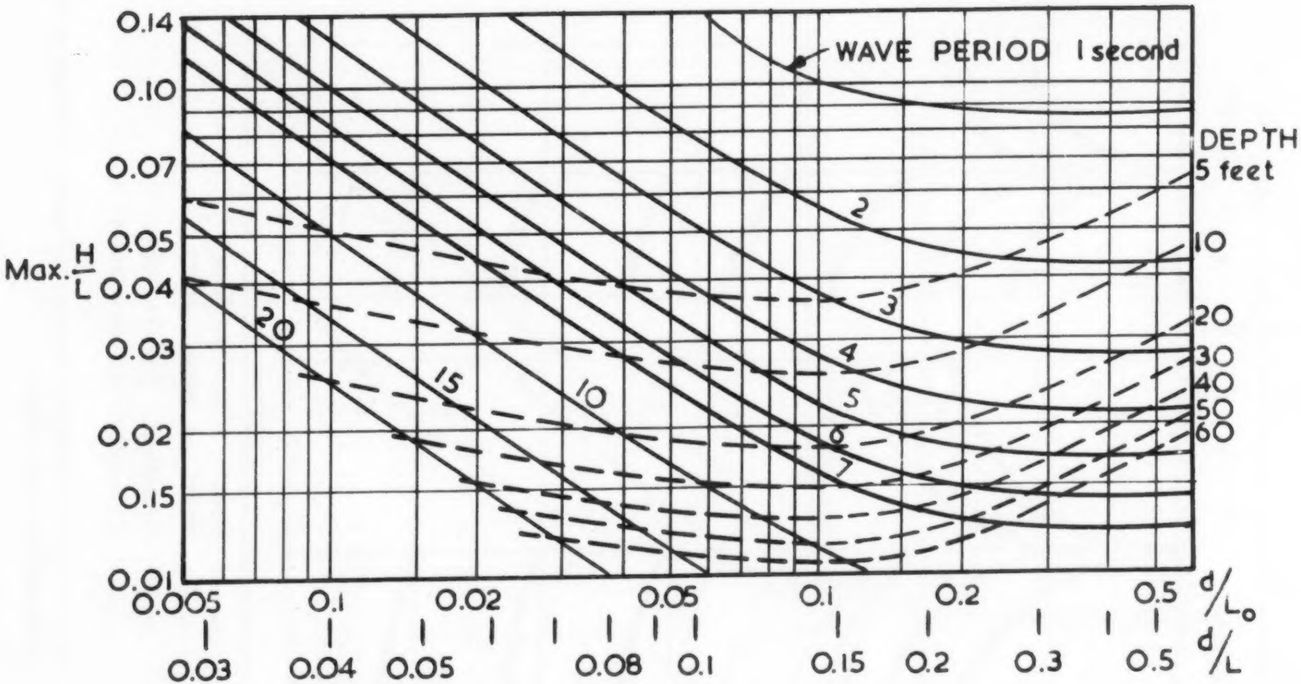

Fig. 1. Maximum expected wave steepness for ocean waves of various periods in different depths of water.

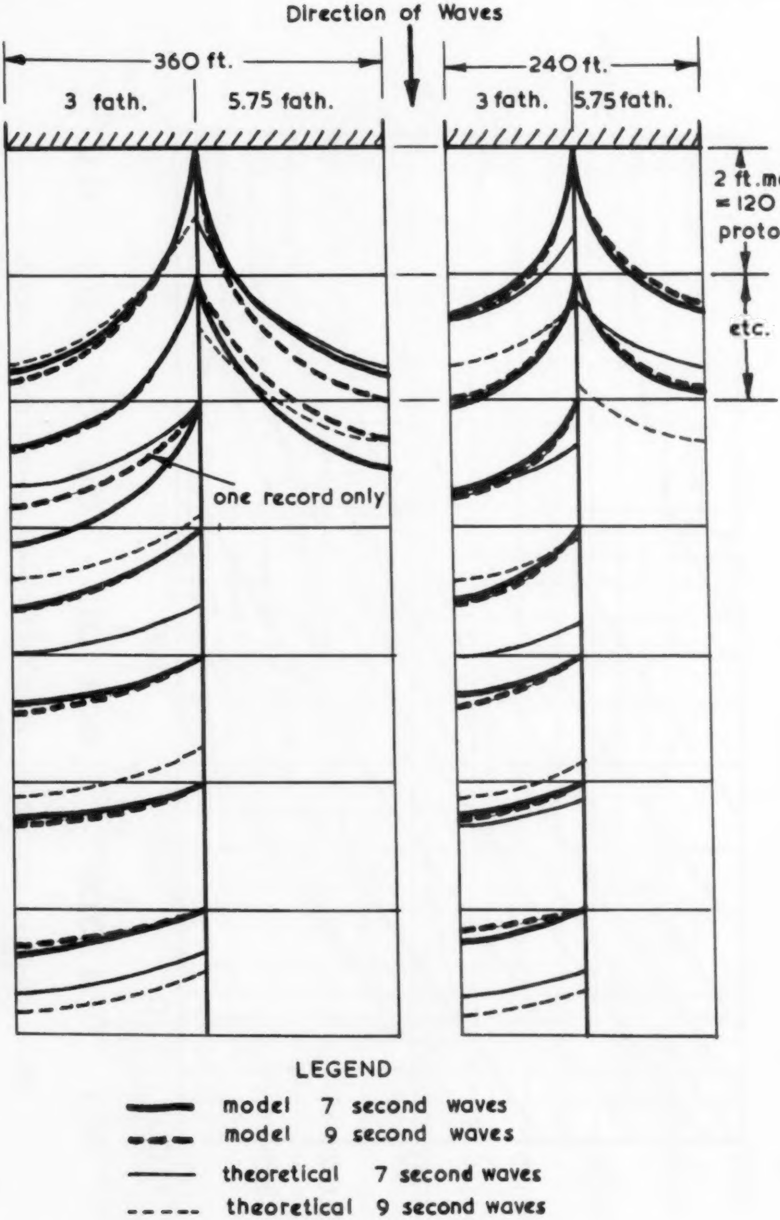

Fig. 2. Wave traces behind the breakwaters for waves of various periods height and still water depth.

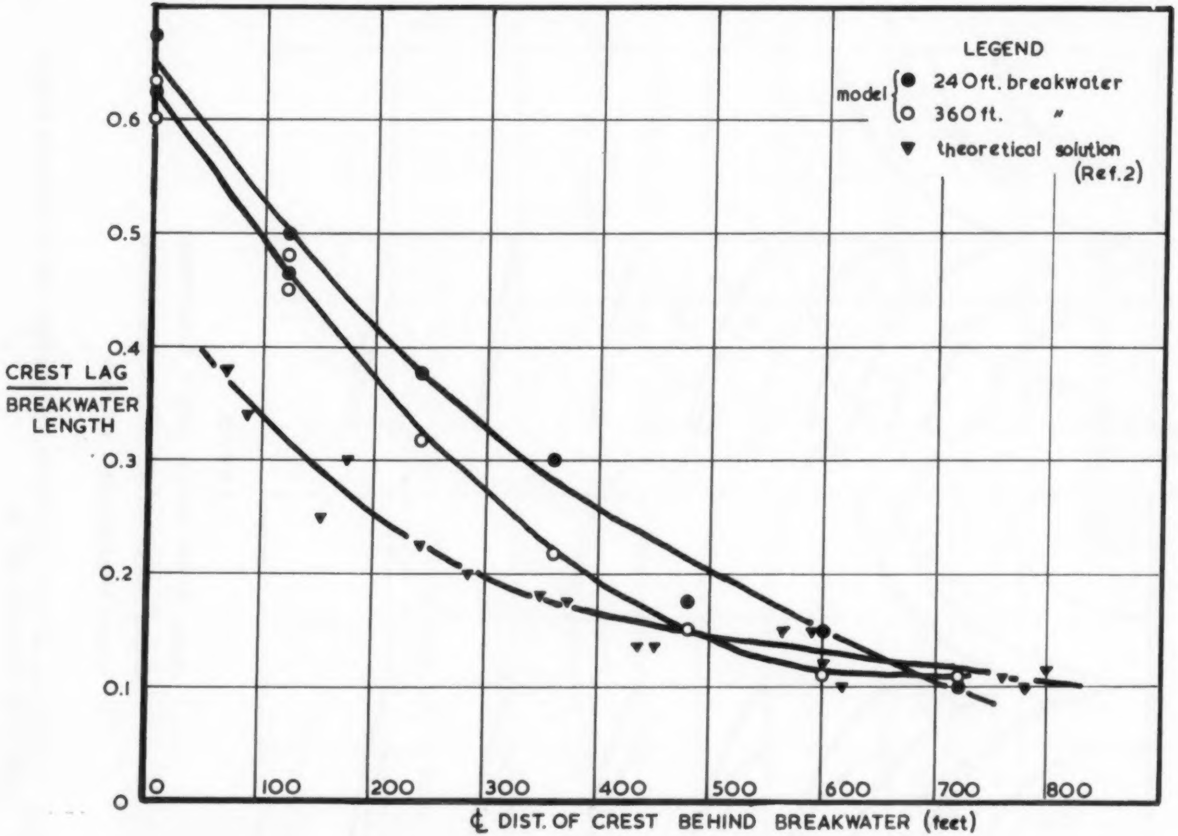

Fig. 3. Values of $\frac{\text{Crest Lag}}{\text{Breakwater Length}}$ at various distances of the crest behind the breakwater as measured on its centre line.

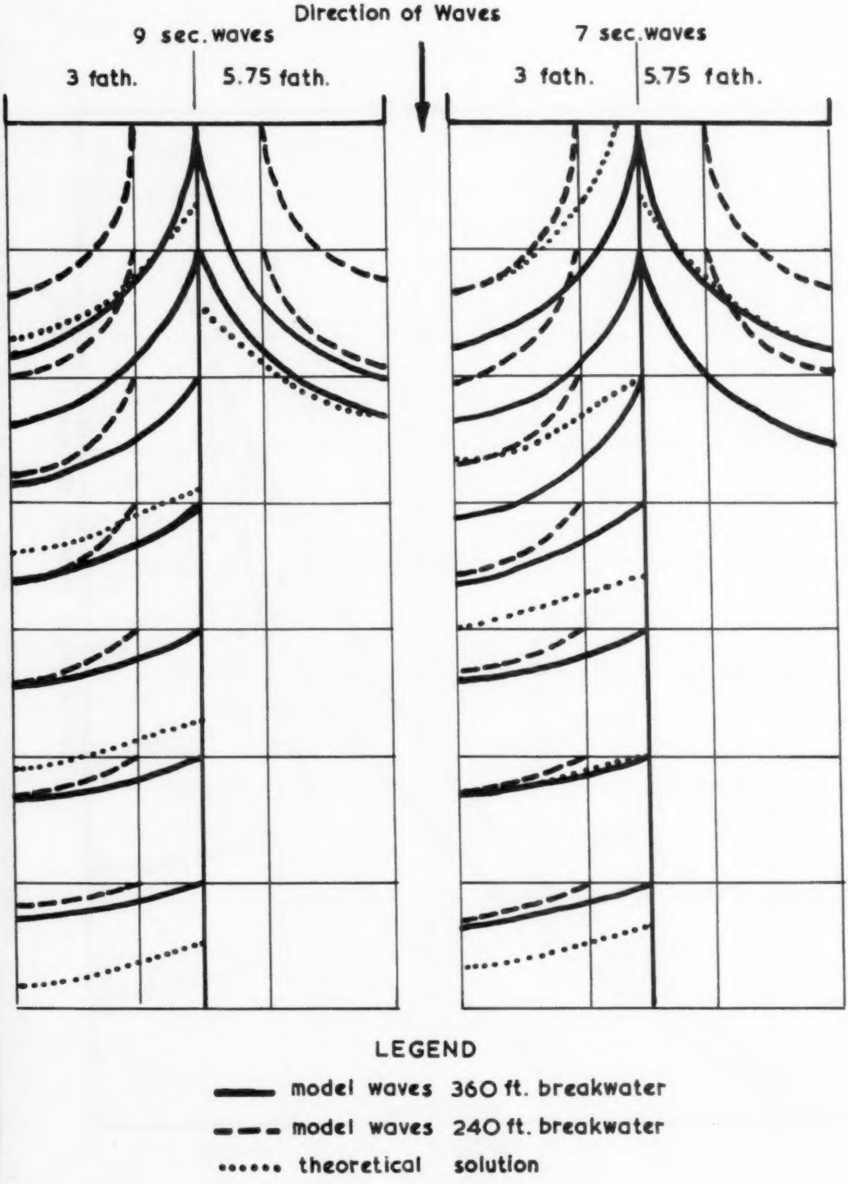

Fig. 4. Comparison of wave patterns behind breakwaters of different length, the ends of which coincide.

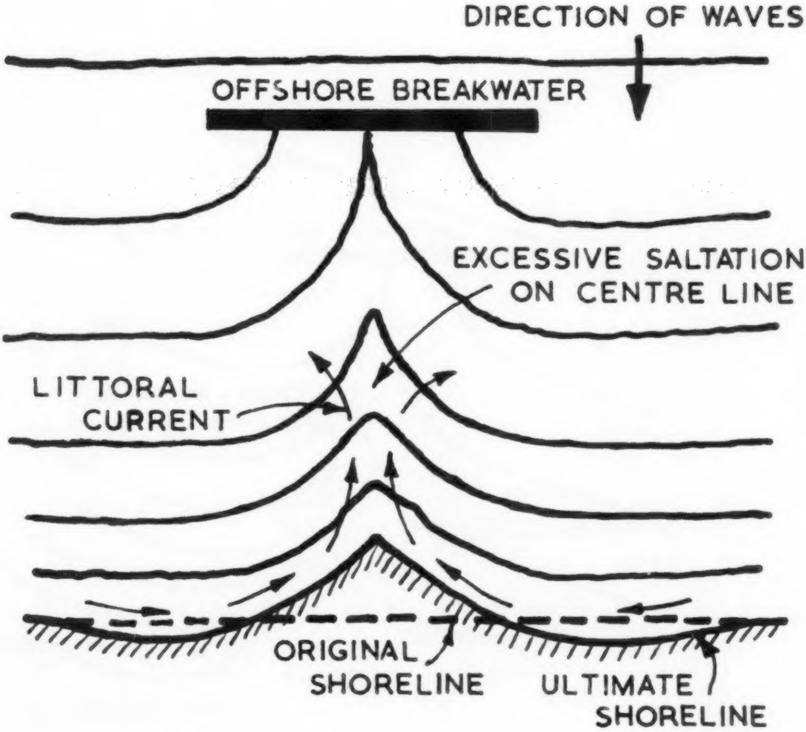

Fig. 5. Change in shoreline due to a breakwater at an intermediate distance offshore and waves approaching at right angles to the coast.

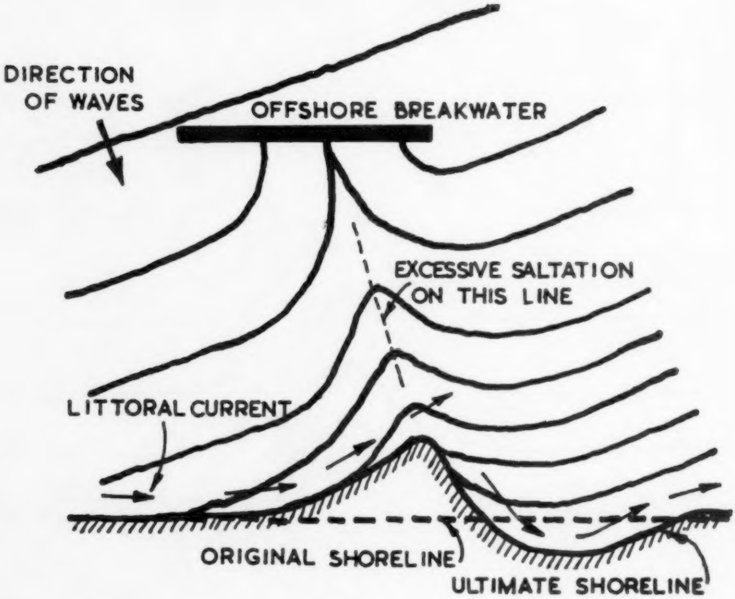

Fig. 6. Change in shoreline due to an offshore breakwater on a coast where littoral drift already exists.

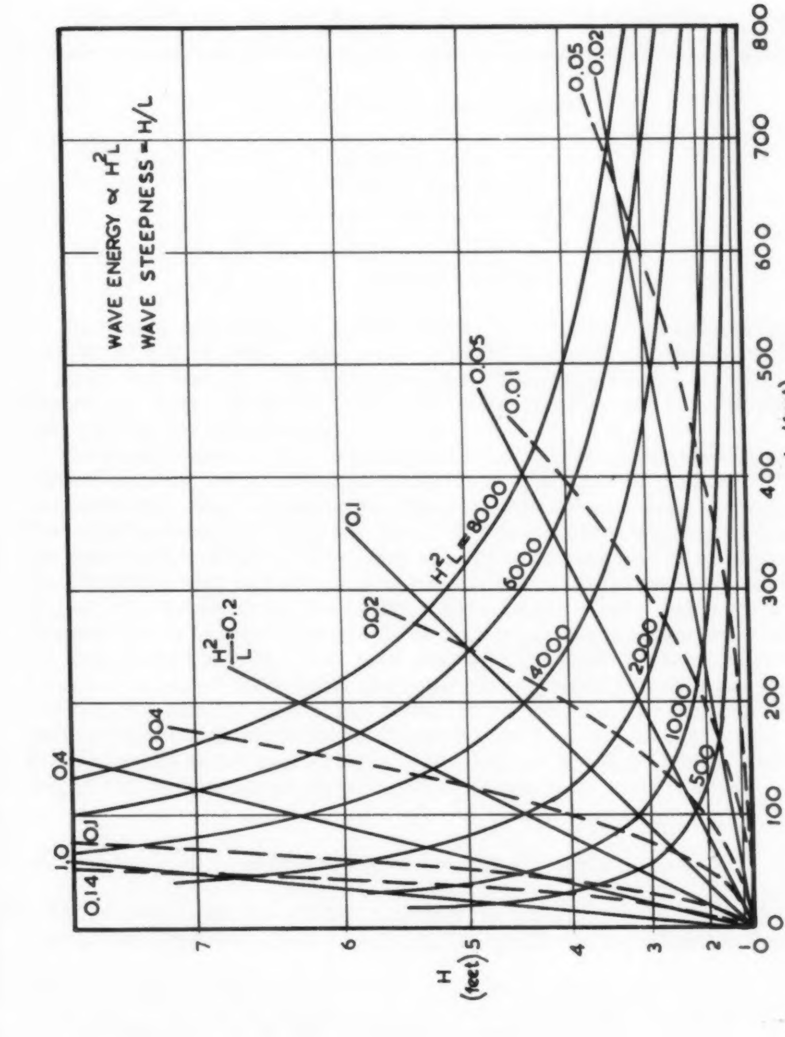

Fig. 7. Variations of H^2L, H/L and H^2/L for different values of H and L.

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GREAT LAKES HARBORS*

Edwin W. Nelson[1]
(Proc. Paper 1369)

INTRODUCTION

This paper discusses the improvements in Great Lakes channels and harbors which will be required as a result of the projected completion of the St. Lawrence Seaway. The discussion covers both the connecting channels between the lakes as well as the harbor channels. Pier and breakwater reconstruction is also discussed.

The most extensive improvements ever undertaken at one time in connection with the Great Lakes navigation system are now in progress. These improvements consist generally of deepening the Great Lakes Connecting Channels between Lake Erie and Lakes Huron, Michigan and Superior and the construction of the St. Lawrence Seaway from Montreal to Lake Erie. The increased size and and draft of vessels which will utilize the improved through channels will require increased depths and larger maneuver areas in Great Lakes harbors. Extensive improvement and reconstruction of existing docks and wharves as well as provision of additional port and terminal facilities will also be required. The purpose of this paper is to indicate in a general manner the overall harbor problems related to the development and maintenance of United States Great Lakes harbors as may be required to accommodate the type and volume of vessel traffic which will utilize the deep draft through channels now under construction.

Improvement of Through Channels

The Great Lakes Connecting Channels, hereafter referred to as the Connecting Channels, consist of the St. Marys River, including the Soo Locks

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at Sault Ste. Marie, between Lakes Superior and Huron; the shoals in the vicinity of the Straits of Mackinac; and the St. Clair River, Lake St. Clair, and the Detroit River between Lakes Huron and Erie.

Present controlling depths in the Connecting Channels are 25 feet in downbound channels and 21 feet in upbound channels when the ruling lake level is at low water datum. Additional deepening of the Connecting Channels has been authorized and construction has commenced to provide a controlling depth of 27 feet in both downbound and upbound channels when the ruling lake level is at low water datum. One lock at the Soo, the MacArthur Lock, has a depth of 31 feet over the sills which is more than adequate for the Connecting Channels depth now being provided. Present safe controlling drafts in the Connecting Channels for bulk lake freighters are considered to be 22.3 feet in downbound and 18 feet in upbound channels when the ruling lake level is at low water datum, whereas in about 5 years after the deepening is completed it will be 25.5 feet for both downbound and upbound traffic. This will provide generally in the Connecting Channels an increase in allowable draft of over 3 feet in downbound channels and of about 7.5 feet in upbound channels. Estimates of safe drafts pertain only to the Great Lakes type bulk carriers and do not necessarily pertain to ocean vessels as no studies were made to estimate safe draft allowances for such vessels when underway. The apparent inconsistencies between controlling depths and drafts under present conditions and for the deepened channels are accounted for in the design of the deepened channels in order to obtain the most economical balanced through channel system after studying the physical and navigation conditions in each reach to be deepened and the characteristics of modern bulk carriers. The allowances established between channel depth and vessel draft varies from 1.5 to 4.5 feet in individual reaches of channel.

The St. Lawrence Seaway now under construction by Canada and the United States will provide a controlling depth of 27 feet from Montreal to Lake Erie via the St. Lawrence River, Lake Ontario and the Welland Canal. A depth of 35 feet is now available from Montreal to the sea. Present controlling depth in the St. Lawrence River canals and locks is only 14 feet. Consequently available depths for ocean traffic entering the Great Lakes is being increased by 13 feet.

Traffic

Total United States traffic on the Great Lakes including all imports and exports averaged about 222,000,000 net tons annually for the five-year period from 1951 to 1955. For the calendar year 1955 the total was about 239,000,000 net tons. Of this total about 165,000,000 tons passed through some reaches of the Connecting Channels and a little over 4,200,000 tons passed through the St. Lawrence River. Of this latter total about 535,000 tons was overseas traffic to United States Great Lakes ports.

The predominant movements through the Connecting Channels are in iron ore, stone, grain and coal. A large percentage of this traffic will be able to take advantage of the increased depths being provided in the Connecting Channels and will require increased depths at ports and terminals.

The 27-foot St. Lawrence Seaway project will generate a large volume of deep draft traffic. This Seaway traffic will be of two general types, that is, bulk cargo such as iron ore, coal and grain, and general cargo. The bulk cargo will in general be of the same type as is now being handled on the

Great Lakes and will be handled by the same or similar terminal facilities. However, the general cargo type of traffic has heretofore been handled in rather limited amounts on the Great Lakes and greatly expanded terminals will be required to handle the future increase in this traffic.

Great Lakes Vessels

Major changes have taken place in recent years in the character of the United States bulk cargo fleet. Prior to World War II the fleet was comprised generally of vessels of from 450 to 600 feet long, 55 to 65-foot beam, with maximum drafts of 21 or 22 feet and with carrying capacities of from 8,000 to 13,000 net tons. In 1942 and 1943, 21 new vessels were constructed from 600 to 625 feet long and 60 to 67-foot beam with maximum drafts of 24.5 feet and with carrying capacities of from 17,000 to 20,000 net tons. Contacts with vessel interests in 1945, in connection with a study of the Connecting Channels, did not indicate any plans for construction of vessels larger than those built in 1942 and 1943. However, construction of the WILFRED SYKES by the Inland Steel Company in 1949 instituted a new era in the type of Great Lakes vessels. This vessel is 678 feet long and 70 feet wide, has a maximum draft of about 27 feet and with cargo carrying capacity of about 23,000 net tons. Since that time 18 large bulk freighters have been built and 8 ocean freighters have been converted for Great Lakes service. These 26 vessels are from 602 to 710 feet long, from 67 to 75 feet wide, with maximum drafts from 24.5 to 27 feet and with carrying capacities of from about 16,000 to 25,000 net tons. This trend in building larger vessels is continuing as evidenced by two more large vessels, 710 feet and 729 feet long, being constructed for service in 1958.

Upon completion of the 27-foot St. Lawrence Seaway the large lake type bulk freighter can be expected to be used in traffic between Greak Lakes and St. Lawrence River ports as well as in inter-lake traffic. It is also expected that a large volume of overseas traffic into the Great Lakes will be in deep draft vessels because of the inherent economies in operation of larger vessels.

Harbors

The discussion herein relates only to United States harbors on the Great Lakes. There are few natural harbors on the United States shores of the Great Lakes. The lower reaches of tributary streams and to a lesser extent the lee sides of islands, coves and bays afforded protection for the first small shallow draft commercial vessels to ply these waters. With the continuing increase in commerce coupled with the development of larger, deeper draft vessels, there was carried out a continuing program of harbor construction to accommodate the fleet.

There are two general phases of port development. One is to provide a safe entrance, inner channels for all traffic, room for turning and maneuvering vessels within the harbor and in some cases sheltered anchorage areas for general use. These facilities are usually provided by the Federal Government except in the case where the harbor serves a single privately-owned establishment. The other phase is to develop the port and terminal facilities for mooring, loading, unloading and servicing vessels. These facilities are provided locally by private interests or local agencies.

There are two general types of harbor entrances on the Great Lakes, namely parallel piers and arrowhead breakwaters so-called because the breakwaters converge lakeward toward the entrance in the form of an arrow. There are numerous variations of this type breakwater system depending upon locality with respect to exposure from the lake and many other factors.

The earliest form of harbor construction consisted of construction of parallel piers at river mouths, not only to provide protection for vessels at the entrance but to overcome the shoaling which occurred near the shoreline from the sediment or silt load in the stream and also in many cases to overcome the shoaling occurring at the harbor entrance due to littoral drift. Entrances of this type are still in existence at active commercial harbors, particularly on Lake Michigan.

It was found that the parallel pier type of entrance did not provide much stilling effect on storm waves entering directly between the piers and consideration was therefore given to other types of protection at harbor entrances. This lead to design of the arrowhead type of converging breakwater with a large expanded area inside the entrance which had a desirable stilling effect because of the diffraction of the waves in the expanded area. There are many variations in the layout of these outer breakwaters. In some cases such as at Superior, Wisconsin and Muskegon, Michigan the converging breakwaters serve only to provide an entry from which vessels pass between parallel piers at the shoreline to enter the inner harbor. At other harbors such as at Milwaukee, Wisconsin, and Cleveland, Ohio, the outer breakwaters extend for several miles in deep water generally parallel to the shore with an entry opposite parallel piers protecting the river mouths. Such breakwaters, in addition to providing a safe entry, also provide protection for development of port and terminal facilities along the shore to supplement those in the inner river channels.

In general the outer end of harbor structures should be in depth of water at least equal to improved channel depth for the harbor to reduce the cost of maintenance dredging of littoral drift material and also to eliminate the hazard to navigation of shoals forming lakeward of the entrance. In a few cases, particularly where the lake bottom slope is quite flat and shoal water extends for a considerable distance into the lake, such as at Green Bay, Wisconsin, Bay City, Michigan, and Toledo, Ohio, it is neither practical nor economical to extend harbor structures into the lake to the depths required for navigation and dredging must be relied upon to provide adequate channel depths lakeward of the mouth of the river. Also in these cases protective structures are not required at river mouths as the extensive shoals break the force of waves before they reach the shore which obviates the need for such structures.

There are 57 federally improved harbors on the Great Lakes with project depths of 18 feet or more. Of these 57 harbors, 3 have depths of 26 feet, 12 of 25 feet, 5 of from 22 to 24 feet, 27 of from 20 to 21 feet and 11 of 18 feet. The deeper harbors are used predominantly for iron ore, stone, grain and coal. In addition to the federally improved harbors there are 9 deep draft private harbors used predominantly for shipment of iron ore and stone and for receipt of iron ore, stone and coal.

The federal harbors on the Great Lakes have in general been developed pregressively from their earliest concept. The present were designed to

accommodate the vessel fleet as constituted some 30 years ago. At that time vessel lengths were limited to about 600 feet with maximum drafts of 21 or 22 feet. Many of the present harbor structures, particularly the parallel piers at river mouths, were designed for channel depths several feet less than are even now provided. With few exceptions, structures at all major harbors were completed at least 20 or 25 years ago and were not designed for channel depths in excess of those now existing in the Connecting Channels.

There are at least 47 vessels in the present United States fleet which must now load at reduced draft except when the lakes are at extreme high stages. Even then a few of these vessels would be required to load at less than maximum allowable drafts. Upon completion of deepening the Connecting Channels increased depths will be required in harbors in order that these vessels can take full advantage of the deepened through channels. The 27-foot St. Lawrence Seaway will also generate deep draft traffic which cannot be accommodated in existing harbors.

The need for increasing harbor channel depths was recognized in studies made in connection with the Corps of Engineers report which recommended the present deepening of the Connecting Channels. By resolutions of both the Senate and House of Representatives Committees on Public Works, dated 18 May 1956 and 27 June 1956, respectively, the Corps of Engineers were requested to consider the advisability of further improvements of the harbors on the Great Lakes in the interest of present and prospective deep draft commerce. The Committees also directed that in the studies due regard be given to the scheduled time of completion of the St. Lawrence Seaway and of the Connecting Channels between the lakes.

Extensive engineering and transportation economics studies are required to determine the harbor improvements which will be economically justified. These studies are underway but in no case have advanced to the stage where specific recommendations can be made at this time. However, it is readily apparent that in consideration of the large volume of traffic involved and the savings in transportation which can be effected through loading of vessels to materially increased drafts that increased depths commensurate with those being provided in the Connecting Channels and in the St. Lawrence Seaway can be justified at many of the more important harbors. At other harbors or in certain portions of more important harbors increased depths of something less than the through channels controlling depth of 27 feet may be all that can be justified.

The actual increase in harbor channel depths which can be justified will require detail study for each harbor as well as for various reaches in certain harbors. It is not expected that all commercial harbors will require deeper channels. Where deepening is required and justified the increase in depth may be as much as 6 or 7 feet although in some cases it will be only of the magnitude of 1 or 2 feet.

It is not expected that in connection with harbor deepening any extensive problems will be encountered in maintaining the stability of the outer breakwater structures which are generally of stone filled timber crib, rubble mound or cellular steel sheet pile construction. However, it is expected that improvement of entrances at some major harbors will be required to accommodate the increasing number of the larger vessels which need more maneuver room than the older smaller vessels for which the harbors were designed. In this regard model studies are now underway for the entry at

Superior Harbor, Wisconsin, and are planned for early commencement for Conneaut Harbor, Ohio.

The Federal parallel pier and revetment type entrance structures present more of a problem in connection with channel deepening. Many of these structures consist of timber cribs and pile piers designed for channel depths even less than present depths. Also because of their age many of them require reinforcement or reconstruction to prevent failure at this time. No particular difficulty is anticipated in accomplishing any necessary repairs or stabilization because of deeper channels. Repairs have been accomplished by facing structures with steel sheet piles with tie rods and anchorages as required to prevent lateral movement. Maximum depths considered along these piers are such that standard sections of Z type sheet piles are adequate for such repairs.

The Federal Government assumes no responsibility for protection or reinforcement of private or municipally-owned piers, revetments or docks which may be endangered due to deepening adjacent channels or due to the need for similar deepening for berthing vessels which is also a responsibility of local interests. If fact, one of the conditions to the Federal Government undertaking deepening of harbor channels would be a requirement that local interests would deepen vessel berths at loading and unloading docks to assure that full advantage could be taken of the channel deepening. This would apply specifically to docks which would handle the commerce which formed the basis for determining the transportation savings required to justify the channel deepening.

Increased channel depths in harbors are considered only where desired and requested by local interests. They should in turn be prepared to determine the condition of existing docks and the effect of channel deepening thereon, to make estimates of cost of any necessary dock repairs or reconstruction required to maintain the stability of the docks and to accomplish such work concurrently with or prior to the channel deepening.

Many of the loading docks are built on pile foundations whose stability would probably not be affected by the relatively small amount of deepening which will be required at most of those docks. However, the large number of earth fill docks with the fill retained by timber or steel sheet pile bulkheads should be critically examined in each case where harbor deepening is desired to determine if the dock design is adequate under the new conditions with increased channel depths.

There are many docks and pile bulkheads along reaches of channel which certain local interests may desire deepened but which bulkheads serve merely to protect the owner's property which would receive no benefit from the channel deepening. In such a case the Federal Government would assume no responsibility for the stability of the structure and would normally proceed with the channel deepening only if satisfactory arrangements were made by local interests to maintain the structure such that the Federal channel would be protected.

No detail surveys have been made of the effect of channel deepening on private pier and dock structures. However, in studying individual harbor channels this matter will be considered in cooperation with local interests in order to determine what the effect will be on existing docks. Any new docks to be constructed should evaluate the probability of future channel deepening to insure to the extent practicable that future operations will not endanger their stability.

The matter of local responsibility for bulkheads and docks along Federal channels which are desired to be deepened by local interests may be more critical at some of the shallower harbors with present depths of 18 to 21 feet than in the present deeper draft harbors because of the increase in depth which may be desired at such shallower harbors. Also many of the smaller harbors have not continued progressively in the development and maintenance of modern docks for deep draft traffic to the extent that has been done in major ports to accommodate existing traffic.

CONCLUSION

Deepening harbor channels to terminal facilities to accommodate the deep draft traffic through the deepened Great Lakes Connecting Channels and through the 27-foot St. Lawrence Seaway will require critical examination of all breakwaters, piers, docks and bulkheads adjacent to channels proposed for deepening.

For the structures for which the Federal Government has responsibility it appears that the outer breakwater structures will require only minor alterations or reinforcement at few localities. However, extensive reinforcement may be required in some cases for parallel pier structures due to their age and present state of deterioration as well as due to the relatively shallow depths for which they were designed.

All non-Federal owners of docks, bulkheads and piers are responsible for such reinforcement or reconstruction of their water front structures as may be required in connection with deepening adjacent channels. Studies have not advanced to the point that the scope or magnitude of this problem can be estimated. However, the program could be extensive and local interests should consider it in their planning for deeper harbor channels.

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OPERATION OF MISSOURI RIVER MAIN STEM RESERVOIRS

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INTRODUCTION

In the Great Plains area of the Missouri River Basin where some of the nation's first irrigation was nurtured and where water rights have been defended at the point of a gun, reservoir storage projects on a massive scale are today taking some of the random variations out of nature's pattern of water supply.

No other place in the United States, with the possible exception of limited areas of the semi-arid southwest, has the water resource presented such a variety of problems to blunt economic progress and stability. These problems run the gamut of disastrous floods, severe drought cycles, limited water supply for municipal and industrial growth; alternate high stream flows and dry river beds, blanket wind erosion of farmlands and erosion by water.

This vast section of our country stretches 1,300 air-line miles from Cut Bank, Montana, to St. Louis, Missouri, a 600-mile wide belt through which courses the unpredictable Missouri River. Three of the Missouri's major tributaries each drain areas larger than the Tennessee Valley. Described by some geographers a century ago, as the "Great American Desert," this region has risen above many of its adversities to become one of the world's great grain and cattle production areas. But population-wise it still lags far behind the national average. It still maintains the traditional status of an almost wholly agricultural economy, hence is highly sensitive to the vagaries of annual precipitation.

Upon this picture of a regional economy geared to nature's weather pattern, has been superimposed in the last decade a man-made system of river controls and water conservation projects of far-reaching significance. The Basin has been the scene of tremendous construction on a large Federal program of water resources development for the past ten years, and is now witnessing a transition of activity to the operating phase. By the end of 1956

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many of the projects had been completed or sufficiently advanced for initial operation, and the actual benefits for which they were authorized and constructed were beginning to be realized. Progress has been particularly outstanding on the main stem of the Missouri River, where the world's largest existing block of man-made reservoir storage capacity is now well-advanced into initial operation.

Description of the Main Stem Reservoirs

The system of Missouri River main stem reservoirs is a major component of the popularly termed "Pick-Sloan" water resources development and control program for the Missouri River Basin. This program, which was authorized by the Flood Control Act of 1944, includes wide-spread use of several scores of multiple-purpose water storage reservoirs, in combination with other works, to provide coordinated water service and control on the Missouri River and its tributaries. The Corps of Engineers, and the Bureau of Reclamation are the Federal agencies assigned the major construction works of this program. Companion programs by the Department of Agriculture and other agencies round out the Basin resource program.

The Missouri River Main Stem Reservoir System, itself, consists of a series of six multiple-purpose projects, including four of the world's larger reservoirs, extending 1,100 miles along the main stem of the Missouri River in Montana, North Dakota, South Dakota and Nebraska. Their location is shown on Figure 1, which also indicates the geographic relationship with respect to the downstream navigation channel and flood control levee projects, and with those of the tributary reservoirs which already are completed or nearing completion.

This 6-reservoir main stem system was planned as the backbone and foundation of the entire Basin program. It is the key to successful reconciliation of intra-basin differences, and large-scale development of flood control, irrigation, navigation and hydro power in the Basin. This system of six reservoirs, primarily, is: to reduce flood flows on the Missouri and Mississippi Rivers; to supply water for large-scale irrigation diversion projects in Montana and the Dakotas; to regulate the remaining flow for water supply, stream sanitation and open-channel navigation on the Missouri River; and to provide for large scale hydro-power generation at the dams. Recreation, fish and wildlife and other secondary purposes also are to be served. Gross storage capacity of the reservoir system will be as follows:

Fort Peck	19.4 million acre-feet
Garrison	23.0 million acre-feet
Oahe	23.6 million acre-feet
Big Bend	1.6 million acre-feet
Fort Randall	6.3 million acre-feet
Gavins Point	.5 million acre-feet
	74.4 million acre-feet

The power plants when completed will have a total installed capacity of 1-3/4 million kilowatts, which with interconnecting transmission lines will constitute

a coordinated central source of electrical generation for a large region in the Dakotas, Nebraska, eastern Montana and western Minnesota and Iowa.

These main stem reservoirs are being constructed by the Corps of Engineers. Construction had advanced sufficiently for Fort Randall Reservoir to be placed in initial operation in 1953, when it was teamed up with the earlier Fort Peck Reservoir which had been in operation since 1938. Garrison Reservoir came into initial operation in 1954, and Gavins Point in 1955. Oahe is currently scheduled for initial storage impoundment in 1959, and Big Bend is planned for initial operation early in the 1960's. By the end of 1956, the reservoir storage capacity available for actual operation totalled approximately 50 million acre-feet; and the name-plate rating of power units actually on the line totalled 712,000 kw. Electrical interconnection of the power plants was accomplished by Bureau of Reclamation backbone transmission lines in June 1956, as the final step permitting completely integrated main stem system operation.

Operating Procedures and Arrangements

Operation of the Missouri River main stem reservoirs involves several elements that tend to make it a very complex problem, and pose unusual requirements for coordination. Several special procedures and arrangements have had to be devised.

Complicating Factors

The principal unusual complications that must be overcome in actual operations fall into six general categories.

The individual reservoirs on the main stem have inter-related effects upon each other, and must be coordinated into an integrated over-all system operation, while also serving individual requirements at each project site. The numerous tributary reservoirs also enter into the picture, and must be properly accounted for in main stem operations. Thus there is a considerable requirement for coordination solely because of the number of reservoirs involved.

This problem is further complicated by the fact that the main stem reservoirs themselves were planned and designed for joint use of their storage capacity to serve five major primary functions, and several other important secondary functions. There are no separate individual storage allocations, except for comparatively small exclusive allocations at the top of the reservoirs for use only in the very largest floods, and the inactive storage in the minimum dead storage pools. Instead, all five major functions with their diverse individual requirements, as well as the secondary functions with their own special needs, are to be served from joint use of common storage. As an example of some of the diversities which need to be reconciled, downstream releases need to be substantially higher during the 8-month open-channel navigation season than is safe when the river is frozen in wintertime, and irrigation demands occur only in late spring, summer, and early autumn; while the demands for power to be generated with the reservoir releases are high the year-around, with peaks in both mid-summer and mid-winter.

The sheer size of the main stem reservoir system, with active storage capacity equal to twice the normal annual runoff, also poses somewhat of an unusual problem. The critical draw-down period for extreme low flows is a

decade long, instead of the one or two years common at most reservoirs. Customary use of the usual annual rule-curve therefore is precluded, and other media are needed as a long range guide during draw-down periods.

The water supply available for regulation by the main stem reservoirs varies widely within any year, from year to year, and even from decade to decade. For example, the annual runoff from the Missouri Basin above Sioux City, Iowa, during the 1898-1956 record period has varied as follows:

median year	25,000,000 acre-feet
minimum year	11,400,000 acre-feet
maximum year	36,700,000 acre-feet

Variations between the runoff in successive years amount to 5,000,000 acre-feet fairly frequently, and have exceeded 10,000,000 acre-feet. During one entire decade the water supply averaged only 60 percent of normal. Furthermore, the runoff is only partially predictable in any year, except on an extremely short term basis of a few days or weeks. Since the main stem system is intended to utilize practically all of the flow of the river except in the very wettest years, and is intended to safely control the runoff in even the wettest years, the vagaries of the water supply present some rather troublesome problems.

Geographically, the area all the way from western Montana to St. Louis and on down the Mississippi River, is involved in operation of the Missouri River main stem reservoirs. The very hugeness of this area, and the diversities of weather and climate encompassed, present some serious operating problems on this score alone.

As might be expected with the great geographic area and the number of different functions, there are a large number of agencies and interests directly concerned with one aspect or another of main stem reservoir operations. Even the single function of power is a divided responsibility—with the Corps of Engineers operating the power plants to produce the hydro power, and the Bureau of Reclamation operating the transmission lines and marketing the power. Recognition and reconciliation of the diverse viewpoints and rights of all these interests into coordinated operations for the mutual benefit of all poses a distinctly unique problem.

The foregoing, collectively, add up to quite a complex situation. However, actual experience is demonstrating that like for any other complex problem effective procedures can be evolved to reduce it to workable proportions.

Principal Media of Coordination

Three principal media have been evolved for handling the Missouri River main stem operating problem: (1) a special advance Annual Operating Plan procedure; (2) a special Coordinating Committee of State and Federal Agency representatives; and (3) a special Reservoir Control Center establishment. Supplemented by additional arrangements for handling details between individual agencies—the most important of which are special arrangements for full continuous coordination between water and power operating organizations of the Corps of Engineers and Bureau of Reclamation—these media are functioning very effectively to overcome the basic complexities of the problem.

The present arrangements were evolved over a period of ten years, beginning immediately after authorization of the Pick-Sloan plan in 1944. Background conditions were favorable. Effective leadership and guidance by the Missouri River States Committee and the Missouri Basin Inter-Agency Committee had led to the establishment of procedures for cooperation and coordination of the many basin interests during the early years of project planning and design. A large number of background planning studies for operation of the completed main stem reservoir system were made and generally agreed upon during this period. Also, considerable experience on the Missouri River was being accumulated from the Corps of Engineers' actual operation of the Fort Peck Reservoir. With this background, a special work group of river control and power experts from the Corps of Engineers and Bureau of Reclamation made an exhaustive study of the problem in 1951 and 1952, and came up with many of the fundamentals of the present arrangement. Considerable study was also given by Basin interests and the States to the possible desirability of some type of special inter-State or Federal-State compact arrangement to handle the reservoir operation problems. This study further clarified some of the problems, and thereby contributed considerably to the present arrangements.

Annual Operating Plans

Annual Operating Plans which are used as the basis for scheduling the actual operations of the main stem reservoirs are prepared well in advance of actual operations.

Each year's Annual Operating Plan is presented in a report which thoroughly covers water supply, special operational requirements and limitations, specific water demands and operational objectives for all functions all along the river; and then concludes with specific operating proposals which are "tailor made" for optimum coordination of the available water supply with water demands and operational requirements. These proposals are summarized into detailed month-by-month schedules of storage contents, releases, and power generation rates for each main stem reservoir and the system. In other words, they outline just where the water will be kept in storage at any given time, just what power plants will meet what portions of the system loads at any time, and just how much water will be released from each reservoir at various times throughout the year. Each year's advance operating schedules are prepared for a wide range of possible water supply conditions. Special 5-year advance projections are also made for each year's annual operating plans, to insure proper attention to the long-range requirements for main stem operation.

The Annual Operating Plans are prepared in August of each year; and may be revised, if unforeseen conditions develop, prior to the next regular revision in the following August. The basic water schedules of the current 1956-57 Annual Operating Plan are indicated on Figure 2. This figure shows actual operations through 1956, and indicates the operation schedules for a median-year water supply in 1957. Other complete schedules for 1957 also are available for so-called "Upper Quartile," "Lower Quartile" and "Adverse" year's water supply.

These water schedules such as illustrated by Figure 2, and companion monthly power generation schedules, actually portray only the broad running-average situation at the reservoirs for the various conditions expected to be

encountered. However, they are predicated on specific detailed objectives which are to be met in actual daily operations. For example, the 1956-57 winter releases from Fort Randall and Gavins Point represent the average of varying day-to-day releases necessary to maintain a current wintertime water supply and stream sanitation requirement of 9,600 c.f.s. at Kansas City; while the higher winter releases from the upstream reservoirs are phased to meet system firm power requirements without violation of minimum water supply and sanitation requirements at any locality along the river between the reservoirs. Similarly, the 1957 spring and summer release schedules for Fort Randall and Gavins Point indicated on Figure 2 represent the average of varying day-to-day releases necessary to maintain flows of 28,000 c.f.s. at Omaha and 35,000 c.f.s. at Kansas City during the navigation season. The 1957 navigation season would begin in early April and extend through early November in the case of the normal water supply situation upon which this particular chart is predicated. The monthly storage and release schedules at all the reservoirs reflect, in this same general manner, the various specific operational requirements and objectives for all the various functions, all along the river.

The preparation of the Annual Operating Plans is facilitated greatly by conformance with certain basic storage zoning provisions and basic operating "ground rules" which were set up for the main stem reservoirs during their pre-construction planning and design. For example, the bottom inactive storage zones of the reservoirs, once filled, are to remain permanently filled with water. This is to insure the maintenance of minimum power heads, minimum irrigation diversion levels, and minimum pools for recreation, fish and wildlife purposes. Similarly, the top storage reservations for handling of the largest floods are to be reserved exclusively for this purpose.

Also, the following general approach which was developed and generally agreed upon during planning and design of the reservoirs, is observed in setting up the Annual Operating Plans:

<u>First</u>, flood control will be provided for by observation of the requirement that pre-determined upper blocks of storage space in each reservoir will be vacant at the beginning of each year's flood season. (This space is available for annual regulation for flood control <u>and</u> all the conservation uses in every normal and wet year, but must be vacant at the beginning of the next year's flood season).

<u>Second</u>, all irrigation, and other upstream tributary water uses during each year will be allowed for. This allowance also covers the effects of upstream tributary reservoir conservation operations, as anticipated from their advance operating plans.

<u>Third</u>, downstream urban water supply and stream sanitation requirements will be provided for.

<u>Fourth</u>, the remaining water supply available will be regulated in such a manner that the outflow from the lowermost reservoirs conforms to the seasonal requirements of navigation, with internal adjustments within the reservoir system and minor adjustments in over-all releases from the system to provide for the generation of the maximum amount of usable power consistent with the foregoing uses.

<u>Fifth</u>, insofar as possible without serious interference with the foregoing

primary functions, the reservoirs will be operated for maximum benefit to recreation, fish and wildlife and other secondary purposes.

These ground rules are valuable prerequisites to setting up the actual operations provided for in the Annual Operating Plans.

Coordinating Committee

The Coordinating Committee on Missouri River Main Stem Reservoir Operation coordinates and consolidates the viewpoints of all the interests concerned with main stem reservoir operations, so that they may be represented adequately in the Annual Operating Plans and carried into actual operations.

The present Coordinating Committee on Missouri River Main Stem Reservoir Operation may be considered a direct product of the inter-agency coordination which has prevailed in the Missouri Basin since the formulation of the Pick-Sloan program and the establishment of the Missouri River States Committee and the Missouri Basin Inter-Agency Committee. The Federal and State agencies still have the same general interests and responsibilities as in the earlier planning and design phases of the program, and to a considerable extent the same individuals in these agencies who had served on various earlier work groups and subcommittees logically continued as agency representatives in this new phase.

Since inception of the Coordinating Committee in 1953, representatives of the 7 States and the 7 Federal Agencies directly interested in main stem operations have actively participated in its functions. The following are represented on the Committee:

State	Federal Agency
Montana	Corps of Engineers
North Dakota	Bureau of Reclamation
South Dakota	Public Health Service
Nebraska	Federal Power Commission
Iowa	Fish and Wildlife Service
Kansas	Geological Survey
Missouri	Weather Bureau

The State representatives—Governor appointed—are generally the State water engineers.

The Committee functions through general meetings, usually twice a year, and through interim contacts with and reports from the Reservoir Control Center. An annual meeting is held, usually in September, to review tentative Annual Operating Plan schedules which have been prepared during August by the Reservoir Control Center. At this meeting the views of all interests are given full consideration, and a specific set of advance operating plans is agreed upon and finalized by the Committee as a basis for actual operations. A second Committee meeting usually is held in April of each year to review actual operations subsequent to the preceding August meeting, to discuss operations during the remainder of the year based on April 1 forecasts and revise the operating plan if necessary, and to outline operational objectives to be considered by the Reservoir Control Center in setting up the tentative

operating plans for consideration at the September meetings. In addition, special meetings are held to consider possible modifications of previously adopted Annual Operating Plans if unforeseen conditions that might warrant important modifications should arise. Such a special meeting was held on November 1, 1956, in connection with the Mississippi River low water crisis which threatened industry in St. Louis.

Each Committee member is responsible for ascertaining, and fully presenting and supporting the viewpoints of his agency or State at these meetings, and is in an effective position to make the various desires and requirements known and to get them integrated equitably into practical plans for operations.

Reservoir Control Center

The Reservoir Control Center has been set up as the action instrument of all the Federal Agencies and Missouri River States directly interested in operations on the main stem. It has been organized to provide the centralized management and direction of water storage and releases which is required for the main stem reservoirs, but in a manner which coordinates, rather than subrogates, the interests and rights of local and State interests throughout the Basin with those of the various Federal agencies involved.

The Reservoir Control Center acts as the information gatherer and base of activities for the members of the Coordinating Committee. It prepares advance estimates of main stem water supply, consolidates advance estimates of water supply requirements, and drafts up advance Annual Operating Plans for consideration of the Coordinating Committee. Then, after the operating plans are finalized by the Coordinating Committee, the Reservoir Control Center directs actual execution of the details of operation, by preparing and issuing daily schedules of water releases and power generation rates for each of the reservoirs and power plants in the system. It is this latter activity which converts the broad monthly-average schedules of the Annual Operating Plan into the actual day-to-day fulfillment of all the various water and power requirements which it has previously been agreed will be met.

The Reservoir Control Center is manned 365 days a year to do this work. Detailed records are kept of all activities, and the various interests throughout the basin are kept currently informed of actual operations, and of new developments which may be pertinent to operation, or affected by operation, as they arise.

The Reservoir Control Center itself is staffed with a group of expert hydraulic and hydroelectric engineers, specially equipped to meet the unique requirements of Missouri River reservoir operation. It is located in the Missouri River Division Office of the Corps of Engineers in Omaha, Nebraska. The working staff is divided into two main groups; a Reservoir Regulation group and a Power Production group. The activities of these groups are fully self-coordinated within the Reservoir Control Center, and each group maintains continuous working contact with related activities at the projects, at the Corps of Engineers District Offices, at the Bureau of Reclamation's Regional Project and Dispatching offices, at the Weather Bureau and Geological Survey offices, and at offices of many other organizations. The completeness of coordination which is maintained through these working contacts is particularly outstanding.

A tremendous volume of weather, river, reservoir and power datum are

communicated to the Reservoir Control Center, analyzed, and resolved into detailed operating schedules for the reservoirs and power plants. Reliable communications and special equipment for analyses are essential to its job. In addition to conventional engineering and computational equipment, the Reservoir Control Center is equipped with a special basin-wide situation plotting map, special reservoir storage and river discharge control charts, weather teletype and fascimile weather map equipment, and special direct-wire teletype and radio communications with the various projects and the main stem power plants.

One detail that is attracting considerable general interest currently is the gradual conversion of analyses procedures for high speed electronic computer solution. A practical automatic computer application has just been developed, as one item of the Corps of Engineers' national Civil Works Investigation program, for preparation of the detailed schedules for the Annual Operating Plans. In this application, the basic inflow estimates, functional water supply and power requirements, and other operational criteria and limits for each basic set of assumptions to be analyzed are fed into the computer. The computer then automatically develops optimum schedules of storages, releases, and power generation rates for each project in the system, which will fulfill the specified service requirements of all the other functions and maximize the generation of marketable power. Month-by-month solutions for an entire year can be obtained in a single automatic computation. This computer process is also adaptable, and will be converted, for use with shorter time periods such as required in actual day-to-day scheduling of operations. Automatic electronic computations processes are also being used at the present time by the Reservoir Control Center in seasonal water supply forecasting, and for probability analyses; and are to be extended progressively to other major work items.

From the foregoing it may be seen that the Reservoir Control Center is the principal center of activity, and "mainspring," of operation of the main stem reservoirs.

Effectiveness of Arrangements

The first four years of actual system operations has witnessed full conformance with the advance Annual Operating Plan schedules and objectives that had previously been agreed upon by the Coordinating Committee. Serious water supply deficiencies in these years made it impossible to completely satisfy all the desires of all the different special interests involved. This raised some special problems, and the new coordination processes were given a severe test. It was necessary to work out operating program for equitable sharing of the water deficiencies. This was done successfully, and quite good service—though slightly limited—was given each and all functions and interests. As it has actually worked out, the adversities of this period greatly strengthened the position and effectiveness of the Coordinating Committee, and clearly demonstrated the practicality of the Annual Operating Plan procedure, and the Reservoir Control Center arrangement.

Summary of Actual Main Stem Operations

Fort Peck, the first of the main stem reservoirs, was ready for initial operation in 1938. However, only very little water was accumulated the first

4 years because of seriously subnormal runoff in the river. After the subsequent return to normal runoff in 1942 filling proceeded steadily and the reservoir was filled to normal operating levels by 1945. The reservoir continued at normal operating levels through mid-1953. Throughout this period the project was operated primarily to firm up the flows of the lower river during the open water navigation season, and was also very effective for flood control and power generation. The annual operation pattern was characterized by relatively low releases during the 4-month winter ice period and ensuing March-June high water season, followed by higher releases in the summer and autumn periods as required to provide for navigation. Winter releases ordinarily averaged 1,000 to 5,000 c.f.s., and late summer releases usually peaked up to an average of 22,000 to 25,000 c.f.s. Lower river navigation was sustained for the full 8-month open water season except in the drier years.

The completion of Fort Randall for initial operation in 1953, Garrison in 1954, and Gavins Point in 1955 brought an end to this single-project, limited-purpose operating pattern, and marked the beginning of comprehensive multiple-purpose system operation on the Missouri River.

As at Fort Peck 15 years earlier, a subnormal flow period coincided with the time the new reservoirs were ready to be filled. As compared to the normal-year water yield of 25 million acre-feet, the natural water yield from the Basin above Sioux City fell off as follows:

1953	23,000,000 acre-feet
1954	19,000,000 acre-feet
1955	16,000,000 acre-feet
1956	19,000,000 acre-feet

During this same period, the new Boysen, Canyon Ferry and Tiber Reservoirs, in upstream headwater areas, intercepted a total of 3 million acre-feet for their initial filling.

Without the water previously accumulated in Fort Peck, there would have been a really serious situation downstream on the Missouri River. Water supply, stream sanitation and navigation requirements needed to be supplied on the lower river, and at the same time dead storage pools totalling 6.5 million acre-feet needed to be filled to bring power on the line by the required dates at the new projects. Fortunately, the water was available in Fort Peck and, as indicated on Figure 3, it was possible to use some of this previously accumulated storage water to augment the limited natural streamflow, and fill the new reservoirs sufficiently for initial power generation by the time required; while at the same time maintaining adequate discharges for water supply and sanitation, and maintaining navigation on the lower river with only moderate curtailment.

The unusually heavy demand on Fort Peck storage pulled that reservoir down from 13.1 million acre-feet in December 1952 to 5.3 million acre-feet in December 1955. This was uncomfortably close to the minimum drawdown level (4.5 million acre-feet). While this drawdown was rather disturbing to local recreational and power interests it had been agreed upon by all the State and Federal representatives, including State officials of Montana, as fully justified by the circumstances. On the brighter side, the total water storage in the main stem system in December 1955 was 13.5 million acre-feet,

a gain of 1/2 million acre-feet over December 1952. By the end of 1956 the storage contents of the system were increased to 14.0 million acre-feet. This was a modest but important gain, which brought the usable active storage capacity in the 4-reservoir system back up to 3.0 million acre-feet. It is important to note that the main stem reservoirs came through this unusually adverse initial situation without any loss in total water storage, and while maintaining good service, even if not ideal, to all the various functions all along the River.

The prospects are favorable for rapid completion of the initial filling of the main stem reservoir system. In a recent study for the Coordinating Committee, the Reservoir Control Center has found that the annual water demand from the main stem supply from above Sioux City for the next few years will need to average only approximately 18.7 million acre-feet to meet all current basic annual requirements. This includes an average of approximately 0.8 million acre-feet annually for new irrigation and reservoir filling on the upstream tributaries, 0.5 million acre-feet for evaporation from the main stem reservoirs, and 17.4 million acre-feet to supply basic downstream requirements for water supply, stream sanitation and navigation. A total of only 17 million acre-feet a year will be required these next few years to meet tolerably restricted requirements in case of continued seriously subnormal water supply; while 22 million acre-feet will provide the best service that could be desired to all functions. It is readily apparent that a substantial surplus will be available for rapid filling of these reservoirs as soon as the water supply returns to normal levels of 25 million acre-feet a year.

In later years, when irrigation has fully developed, this currently surplus water for a normal water year will no longer be available, and future refilling of the main stem reservoirs will have to be done in wetter than average years. However, the main stem reservoirs will have completed their initial filling well in advance of the scheduled full scale development of the irrigation.

The rate at which the main stem reservoirs will be filled, assuming 1957 and each subsequent year has a normal 25 million acre-foot water supply, is also shown on Figure 3. This forecast, being based on the median-year expectancy, represents the most likely probabilities. Actual filling of these reservoirs may be advanced or retarded somewhat, depending on the actual magnitude and sequence of water supply during this filling period.

Benefits Already Being Realized

The transition into initial system operation of the Missouri River main stem reservoirs has been accompanied by a rapid expansion in benefits.

Flood control on the Missouri River took its greatest single step forward with the bringing of Fort Randall and Garrison into operation in 1953 and 1954. Never again will a great flood from the area above these dams like that in 1952 ravage the lower valley. The local protection works on the lower river and reservoirs on lower basin tributaries will have to be completed before the Missouri River is free from floods, but the main stem reservoirs are already handling one of the worst flood sources.

The initial operation of the main stem reservoirs has been highly successful in maintaining satisfactory flows for water supply, stream sanitation and navigation on the Missouri River. Also, the controlled flow on the Missouri

River has contributed substantially towards maintaining adequate flows in the Mississippi River at St. Louis and below. The actual record of low flow augmentation, as indicated on Figure 4, is a testimonial to the value of these reservoirs. Up to 3/4 of the necessary navigation season flow at Omaha in 1953, 1954, 1955 and 1956 have been supplied from storage in the main stem reservoirs. In October 1956, 50 percent of the total flow in the Mississippi River at St. Louis came from these reservoirs, preventing what otherwise would have been a very serious low flow situation.

Power generation at the main stem plants has been expanding rapidly as the new plants are coming into service. Actual generation at the main stem plants, by calendar years, has increased as follows:

1952	610 million kwh
1953	620 million kwh
1954	820 million kwh
1955	1,400 million kwh
1956	2,280 million kwh

The interconnection of plants in June 1956, which permitted higher releases to be made through the upstream plants in wintertime without waste of water downstream from the system (Fig. 2), brought the year-around system firm capability up to over 400,000 kw. This power is of particular significance and value in the high fuel cost area where it is marketed.

Actual use of main stem water for large scale irrigation awaits the completion of the actual irrigation project works. Meanwhile, expanding tributary water use is being fully allowed for, and the water bank is being filled for future large scale irrigation directly from the Missouri River.

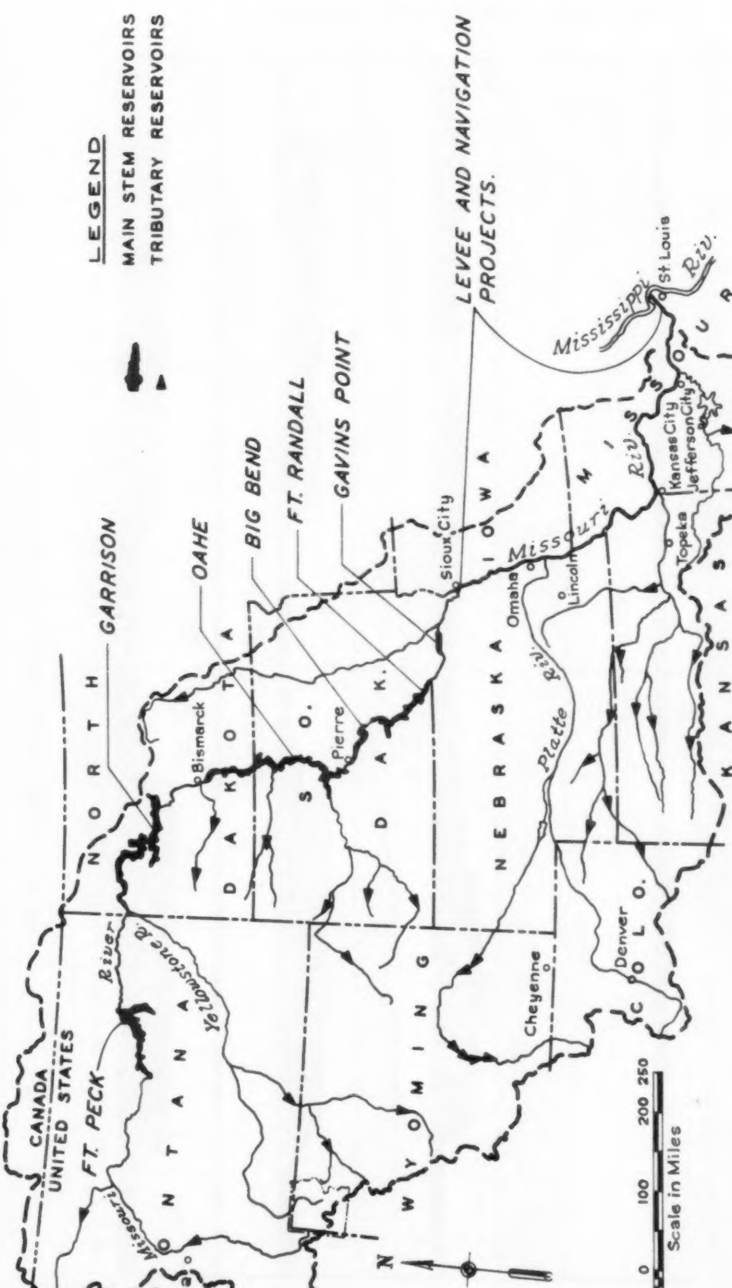
The initial filling of these "Great Lakes of the Missouri" also is greatly expanding the recreational, fishing and hunting opportunities in the region. Residents of the area are particularly well pleased with this situation.

These benefits which have been realized to date, while in themselves of great importance, are actually just a sample of the several-fold greater values to be realized when all the Missouri River main stem reservoirs are completed, and they are filled to normal operating levels.

CONCLUSION

Many complex problems and obstacles had to be solved to bring the Missouri River main stem reservoirs into initial system operation. These problems are all being met and mastered by the procedures and arrangements which have been devised. Actual accomplishments of operation clearly attest to this success, and indicate quite tangibly the great ultimate value of the Missouri River main stem reservoirs, when all are completed, to the region and the nation.

A great many engineers in many agencies have contributed directly to the success in bringing these reservoirs into operation. All these, and the whole profession, can justifiably take great pride with this accomplishment.



MISSOURI RIVER BASIN

Fig. 1—Location Of Main Stem Reservoirs

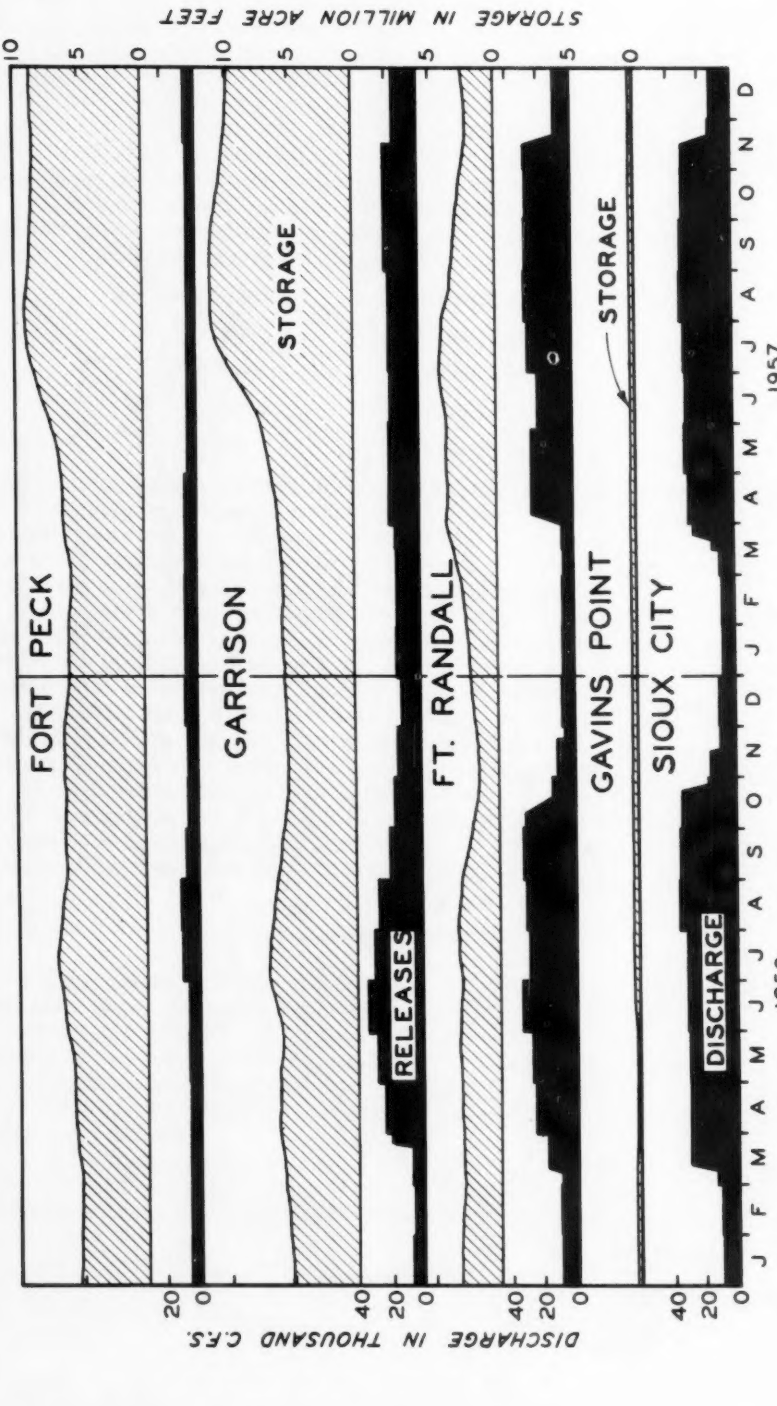


Fig. 2—1956-1957 A.O.P. Water Schedules

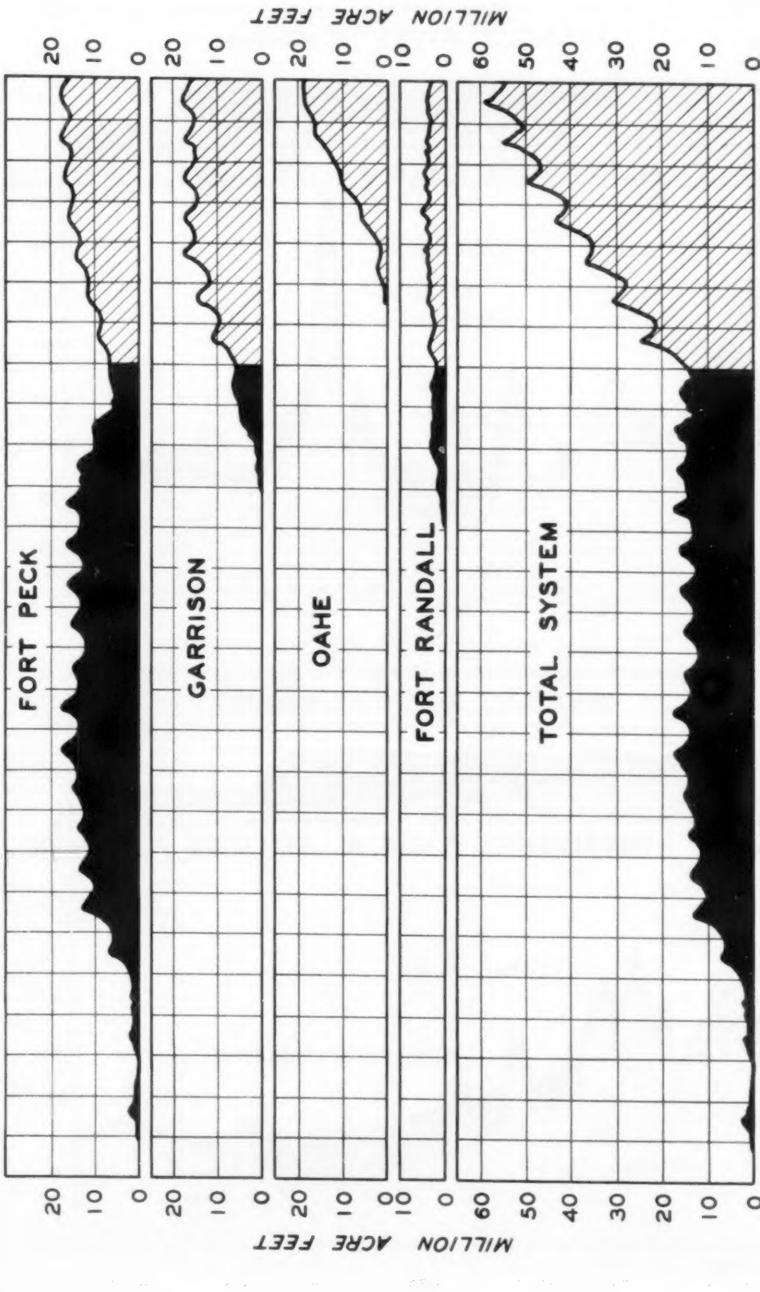


Fig. 3 - Filling of the Reservoirs

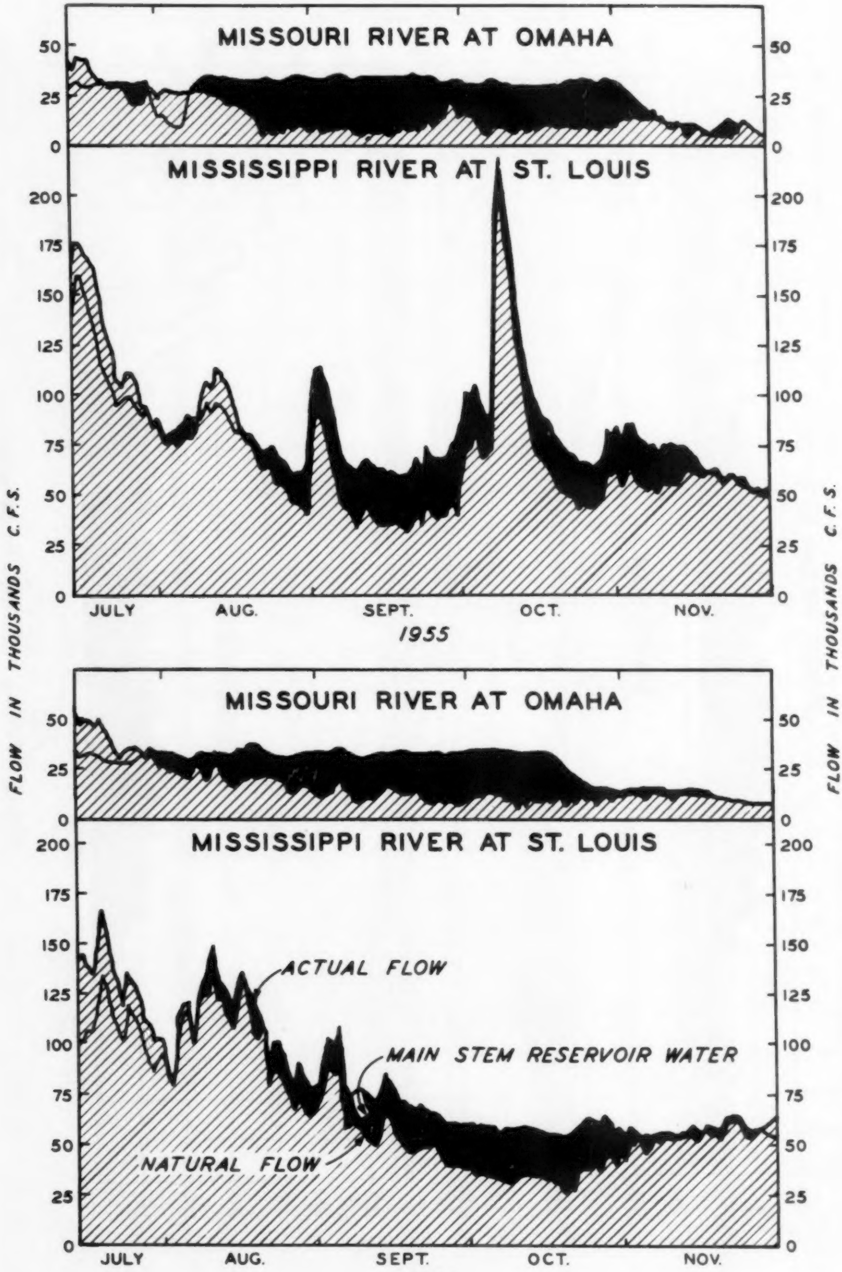


Fig. 4—Low Flow Augmentation

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1381)

Discussion of "ANCHORAGES FOR LARGE TAINTOR GATES"

by A. H. Kenigsberg
(Proc. Paper 1119)

S. H. WEARNE, A.M. ASCE.*—In planning the power intakes for the Macagua no. 1 hydro-electric scheme in Venezuela on the River Caroní, Mr. Buzzell's recent paper and the discussion of the nowadays wider use of Taintor type gates was of considerable interest, as an intake design owing something to Swedish schemes such as Harsprånget was developed for Caroní using these gates measuring 10 by 11 metres face area. The hydraulic development and model tests of this intake design are described in a paper being given by Mr. H. D. Morgan to the July 1957 Congress in Lisbon of the International Association for Hydraulics Research.

Concerning Mr. Kenigsberg's paper, for Macagua no. 1 the anchorage design was developed by the needs of structural economy and also progress of work in a tight and all-important program. After consideration of distributing the thrust, 600 T. on each trunnion, by heavy reinforcement into the mass concrete intake buttresses, the use of A frames of structural steelwork members was adopted at the tender stage—a conventional arrangement as discussed in the paper. The frames were to be supplied with the gates, detailed designs being prepared by tenderers for approval. Preparation of tenders on these lines progressed such that the gate contract would be placed late in 1956, for the design, construction and erection of the first gates to be completed in two years. It was considered essential that the large A frames should be supplied in time for casting into the buttresses during concreting and that grouting of the frames later into boxed-out cavities would not give the required intimate distribution of the thrust. This scheme was, however, reconsidered from the needs of program and it was concluded that there was insufficient time to develop design of the A frames, to fabricate and to ship them.

An alternative arrangement was therefore designed with the trunnions mounted on plate grillages. The grillages comprise a pair of heavy plates coupled by bolts, lying in the plane of the buttress faces distributing the force to reinforcement, arranged so that the grillages could be erected and grouted up after buttress concreting if necessary.

Incidentally, two points of interest arise concerning the name "Tainter" (or "Taintor") gate. Spelling of this name seems to vary. I understand there was a Captain Tainter, or Taintor: can anyone answer who he was and why the name is confused?

Also, since this type of gate is surely not the invention of any recent captain and is widely used, would it not be useful if there was agreement to refer to this type of gate always by the other usual name of radial gate?

(Ed. Note.—In answer to Mr. Wearne's queries, we offer the following reprint of a contribution by Don. H. Mattern, M. ASCE from the June 1950 issue of <u>Civil Engineering</u>:

* Site Engr., Nuclear Power Plant Co., Bradwell, Essex, formerly Caroni Section Leader, Sir William Halcrow & Partners, Consulting Engineers, London, England.

Patent Office Record on Tainter Gate Cited

To the Editor: Mr. Streiff's interesting story in the March issue, page 29, on a record-size radial gate installed in Spain, together with the publication of the second edition of a very excellent hydroelectric handbook, once again brings to our attention the spelling of the word "Tainter." Mr. Streiff uses the spelling "Tainter," while the handbook uses "Taintor"—as it also did in the first edition, 1927.

Since many engineers still prefer to use the word "Tainter" instead of the more modern "radial," we have looked up the patent reports in the *Official Gazette of the United States Patent Office.* We found that the handbook spelling is wrong, and that some of Mr. Streiff's data do not check with the recorded patent information. The following data are presented to get the record straight.

Application for patent No. 344,878 was made on November 16, 1885, by Jeremiah B. Tainter, Menomonee, Wis. The record appears in the listing for July 6, 1886, so that is presumed to be the date when Mr. Tainter actually obtained his patent.

A photostatic copy of the above data is attached for your information and files.

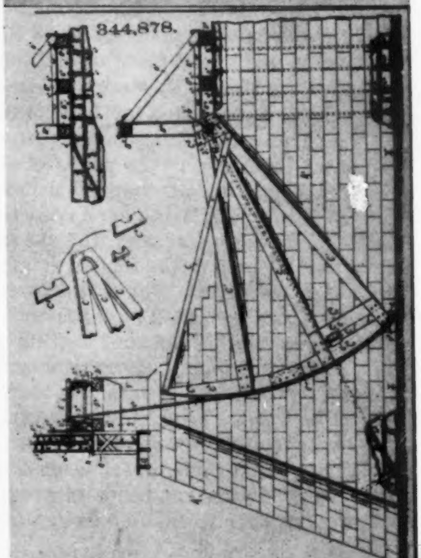
Don H. Mattern, M. ASCE
Knoxville, Tenn.

344,878. SLUICEWAY-GATE. Jeremiah B. Tainter, Menomonee, Wis. Filed Nov. 16, 1885. (No model.)

Claim.—1. The combination, with a sluiceway-gate, of a hoisting apparatus consisting of a frame, D^2, having spirally-grooved chain-drums E' E^2, adapted to be revolved and connected to the sides of said gate, said frame D^2 being movable in a line across said gate, whereby the tension of said chains is equalized, substantially as set forth.

2. In a sluiceway-gate, the combination, with said gate, of a truss-frame, D', supported on piers A' A^2 above said gate, frame D^2, having spirally-grooved drums E' E^2, adapted to be revolved, spirally-grooved guide-drums E^3 E^4, and chains r' r^2, connecting said gate with said drums E' E^2 over said drums E^3 E^4, substantially as set forth.

3. The combination, with a sluiceway-gate, of truss-frame D', supported on the piers of said gate, frame D^2, freely movable along said truss-frame, spirally-grooved drums E' E^2, gears E^5 E^6 on the shaft of said drums and engaged by one common pinion, E^7, on shaft E^8, chains r' r^2, connecting said gate with said drums, and a wheel connected to and adapted to revolve said shaft E^8, consisting of clutch-hub m' m^2, central grooved rim, F', side rims, F^2 F^3, connected to said central rim by rounds c', and a brake-rope, a', substantially as set forth.



4. In a sluiceway-gate, piers A' A^2, provided with straps a', united to the sub-foundation X of said piers by anchor-bolts, truss-frame consisting of main timbers B' B^2 B^3 B^4 and braces B^5, B^6, and B^7, said truss-frame lying across said sluiceway and secured to said piers by straps a^2, and bolts a^3, uniting its ends to said straps a', in combination with a sluiceway-gate consisting of arms C', ribs C^2, facings C^3, and braces C^4, each of said arms C' attached at one end to said ribs, and converging and encompassing bearings b' on said timbers B', substantially as set forth.

5. In a sluiceway the piers whereof diverge from the bed of the sluiceway upward, a sluiceway-gate corresponding with said divergence and provided with self-adjusting guide C^8, adapted to support said gate from side-thrust while being raised and lowered, substantially as set forth.

6. A sluiceway-gate consisting of arms C', ribs C^2, facing C^3, and braces C^4, said arms converging and adapted to encompass bearing b' in truss-timbers B', in combination with guide C^8, adapted to permit the free perpendicular movement of said gate, but to prevent lateral movement, substantially as set forth.

7. A sluiceway-gate consisting of arms C', ribs C^2, facing C^3, and braces C^4, each of said arms C' attached by one end to said ribs and converging and united at their other ends, and adapted to encompass bearing b' on pivotal timbers B', hard-wood bearing-blocks b^2, inserted in said arms to receive the strains of said bearings, and side plates, C^5, adapted to unite said arms and support said bearing-blocks, substantially as set forth.

PHOTOSTAT FROM *Official Gazette of the United States Patent Office* (v. 36, pp. 21–22, July 6, 1886) shows drawing and description of Tainter gate for patent No. 344,878, awarded to Jeremiah B. Tainter, presumably on that date.

Discussion of
"DEVELOPING PORT FACILITIES ON HOUSTON'S SHIP CHANNEL"

by Frank H. Newnam, Jr.
(Proc. Paper 965)

FRANK H. NEWNAM, JR.,[1] M. ASCE.—The discussion papers by Messrs. Brant and Lantz are not only of value to the development of this subject but add much needed information concerning design criteria for general cargo wharf facilities. The surprising feature of the discussions is the high percentage of general concurrence in the criteria developed, for it is believed to be generally accepted that no two sets of conditions are ever exactly the same at two different ports. This point is emphasized because it will always be necessary to carefully consider and study the prevailing conditions in preparing the design for specific facilities.

Since Mr. Brant's discussion concerns the same area at the Port of Houston, however, a few comments of explanation are offered concerning some of the points raised by him. On page 1124-4, fourth paragraph, he properly points out some of the advantages of an apron elevation of 25 feet above mean low water. The writer's study recommended an elevation of +20 feet for the following reasons:

1. Transition could be made within a reasonable distance to increase the height from +18 feet (existing at Wharf 16) to +20 feet, and thus permit the flow of rail and truck traffic between the aprons of adjacent wharves.

2. Subsequent to 1953 there has been a considerable amount of excavation (earth borrow) from this area which greatly reduces the expense of utilizing an apron elevation of +20 feet.

3. The rapid increase in the cost of the substructure that accompanies a higher apron elevation is believed to more than offset the additional cost of earth removal in this area.

4. Sufficiently flat railroad grades could be worked out for the area, utilizing this elevation of +20 feet for the apron. It will be noted that our study recommended a maximum railroad grade of 1 per cent and a maximum desirable degree of curvature of 12.5 degrees (See Table 1).

As a matter of interest, the Port of Houston has subsequently decided to adopt an apron elevation of 18 feet above mean low water for the four new wharves in this area.

The last paragraph on page 1124-4 of Mr. Brant's discussion, points out that only two tracks are recommended in their report for this area. In our visits to, and studies of, other ports, we found that there was no agreement among port operators on this subject. The 50 foot apron width does permit the use of three tracks, which greatly facilitates the handling of cars and is preferred by the Port of Houston, which fact quite frankly influenced the recommendation on this item.

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1. Partner, Lockwood, Andrews, & Newnam, Houston 19, Tex.

It is gratifying to see that the cost estimates of the two reports are remarkably close, when allowance is made for the difference in size of the transit sheds. The difference in area of transit storage sheds is due to a different concept in the arrangement and use of the sheds and particularly in the design loading. Our study recommended a design loading of 600 pounds per square foot (as compared to the 300 pounds per square foot used by Mr. Brant) which materially reduces the required shed area. This and other items of difference indicate the fact that there are normally more than one engineering approach to solve a specific problem.

In the third paragraph of the discussion by Mr. Lantz (page 1124-8), he points out that the recommended apron elevation of 20 feet is some 8 feet higher than the elevation selected for their new facilitiy. This again points up the fact that each specific facility must be designed for its own governing conditions, and Mr. Lantz states some of the conditions at their port that differ from the Port of Houston. If we used the artillery method of "bracketing the target" however, an average between the elevation of 25 feet recommended by Mr. Brant and the 12 feet recommended by Mr. Lantz would be 18-1/2 feet.

It is noted that the cost per berth for the facilities described by Mr. Lantz are considerably less than the estimates listed in our report and the discussion by Mr. Brant. This is undoubtedly due largely to the difference in channel depth and apron height. In designing different waterfront facilities, we are constantly amazed at the rapid increase in cost that accompanies any relatively small increase in apron height and/or channel depth.

For the items not affected by the apron height, such as the transit sheds and lengths of berths, there appears to be close agreement between the dimensions selected at the two ports. On the item of rear loading platform, it is agreed that the width of 30 feet used at the Port of Brownsville is preferable to the width of 20 feet recommended for the Port of Houston, wherever the owner is willing to accept the slight increase in cost.

Undoubtedly, the discussions by Messrs. Brant and Lantz and the design criteria listed therein will be of material benefit to those interested in the planning or designing of waterfront facilities for general cargo.

Discussion of
"RIVERS UNDER INFLUENCE OF TERRESTRIAL ROTATION"

by Gerard Tison, Jr.
(Proc. Paper 1208)

GERARD TISON, JR.[1]—The different applications of the influence of the Coriolis Force on river flow given by the author are quite remarkable.

The observations which were made in some tidal rivers give another confirmation of the theoretical considerations presented by the author. During the flood current the flow is deflected to the right in the northern hemisphere increasing on the left bank the height of the high waters. Likewise during the ebb current the flow is deflected to the right bank causing there higher low waters. A typical example is given by the Delaware Estuary, the range of tide being there in the widest part of the estuary about 0.9 ft. greater along the left bank than on the right. The deflection of the current to the left bank than on the right. The deflection of the current to the left bank during flood and to the right bank during ebb was also observed on nearly straight reaches of the river Elbe and of the river Weser.

On the other hand there exists some analogy between the effect of terrestrial rotation and the effect of a bend on the flow in a river. It is the writers intention to recall that analogy and to show that the assimilation of the two effects is not entirely sound with regard on erosion phenomena.

As a direct result of the existence of the Coriolis acceleration:

$$a_2 = 2\,v\,.\,u \sin f$$

a body moving free on the surface of the earth with a velocity $\underline{v}$, follows a trajectory the radius ρ of which is given by:

$$\frac{v^2}{\rho} = 2\,v\,.\,u \sin f \tag{1}$$

In the northern hemisphere the center of curvature is situated on the right.

In the case of a moving mass of water guided in a straight part of a river, the water particles will not be able to proceed on the line defined by eq. (1). However by effect of terrestrial rotation the mass of water will press on the right bank (North of the equator.). Consequently by reaction this right bank will exerce a force (acting to the left) on the liquid.

Likewise one observes in a bend of a river that the concave bank exerces a comparable action on the fluid particles.

Thus the effect of the terrestrial rotation results comparable with the effect of a bend, the centrum of curvature of which should be situated on the left with reference to the direction of flow (in the northern hemisphere).

Consequently some authors have proposed to replace the effect of terrestrial rotation by a bend with a fictive radius ρ_f given by:

1. Civ. Engr., Hydr. Lab., Univ. of Ghent, Ghent, Belgium.

$$\frac{v^2}{\rho_f} = 2\,v\,.\,u \sin f \tag{2}$$

That assimilation implies that the water elevation along the right bank (in the northern hemisphere), the deflection of the current, the erosion of the right bank and of the bottom along this bank with deposition of sediments on the other bank should be identical with correspondent phenomena observed in a curve of the same river with radius ρ_f determined by (2).

In the case the river presents a real curve with radius ρ_r, it has been proposed that a measure of the combined effect of that curvature and of terrestrial rotation might be given by assuming for that curve a fictive radius as defined by:

$$\frac{v^2}{\rho_f} = \pm \frac{v^2}{\rho_r} + 2\,v\,.\,u \sin f \tag{3}$$

The + sign in eq. (3) corresponds in the northern hemisphere with a real curvature with center on the left bank.

In some cases the fictive curvature will have an opposite sign to that of the real curvature and the main current will be observed along the convex bank of the curve instead of along the concave bank. This indeed has been observed on the Elbe and on the Weser.

However it can easily be shown that the effect of a bend is to make the current dive along the outside bank and resurge along the inside bank.

If one supposes that in a bend of a river succeeding to a straight reach streamlines are parallel to one another and to the bottom, it can easily be seen that the streamlines on the outer side will have to take immediately a curvature, while those on the inner side remain still rectilinear. AB and CD are lines which are tangent to the principal normals of the trajectories (see figure 1); AB is near to the surface while CD is near to the bottom. The points B and D are supposed to be in the same perpendicular on the bottom. Applying now the general equation of hydrodynamics to the lines AB and CD it may be written:

$$z_A + \frac{p_A}{\gamma} = z_B + \frac{p_B}{\gamma} + \frac{1}{g}\int_A^B \frac{v^2}{R}\,dR \tag{4}$$

$$z_C + \frac{p_C}{\gamma} = z_D + \frac{p_D}{\gamma} + \frac{1}{g}\int_C^D \frac{v'^2}{R}\,dR \tag{5}$$

in which z are the elevations above a horizontal reference plane, p are the pressures, v the velocities near the surface, v' those near the bottom, R is the radius of curvature of the trajectories and γ the specific weight of the liquid.

As BD is perpendicular not only on the velocities but also on the accelerations of the liquid, it can be considered as a hydrostatic line, thus:

$$Z_B + \frac{p_B}{\gamma} = Z_D + \frac{p_D}{\gamma} \tag{6}$$

The combination of eqs. (4), (5), (6) yields:

$$\left(Z_A + \frac{p_A}{\gamma}\right) - \left(Z_C + \frac{p_C}{\gamma}\right) = \frac{1}{g}\left[\int_A^B \frac{v^2}{R}\, dR - \int_C^D \frac{v'^2}{R}\, dR\right] \tag{7}$$

Eq. (7) can only be satisfied if the trajectories along the concave bank are diving. Indeed the application of the general equation on the line AC yields:

$$\left(Z_A + \frac{p_A}{\gamma}\right) - \left(Z_C + \frac{p_C}{\gamma}\right) = \frac{1}{g}\int_A^C j_t\, ds \tag{8}$$

where j_t is the projection of the total acceleration on the line AC. As a result of eqs. (7) and (8):

$$\int_A^C j_t\, ds = \int_A^B \frac{v^2}{R}\, d\, R - \int_C^D \frac{v'^2}{R}\, d\, R \tag{9}$$

As the second member of eq. (9) is positive the projection of the total acceleration results acting in the direction of the bottom and the trajectories are consequently diving along the outer bank and the strength of the component producing this diving motion increases with the difference:

$$\int_A^B \frac{v^2}{R}\, d\, R - \int_C^D \frac{v'^2}{R}\, d\, R$$

On the other hand the diving flow in the vicinity of the bottom forces the streamlines of the bottom to the inner bank where the fluid will surge to take the place of the fluid removing from that bank. So a circulatory movement is created which, combined with a general translation movement, results in the wellknown helicoidal movement.

As diving streamlines easier erode the bottom a more important bed erosion is observed along the outer bank.

Considering now a rectilinear part of the river, it has been shown that the rotation of the Earth produces in the fluid an acceleration 2 v . u sin f perpendicular to the bank.

Applying now the same general equation on two lines AB and CD, as it has been done for the bend, one comes to the conclusion that the flow will dive along the right bank in the northern hemisphere. The intensity of the diving will increase with the difference:

$$\frac{1}{g}\int_A^B 2\, v\, .\, u \sin f dR - \frac{1}{g}\int_C^D 2\, v'\, .\, u \sin f\, dR$$

As in the case of a bend the diving flow will be at the origin of a helicoidal movement. The question remains if the effect of a bend and the effect of terrestrial rotation can be assimilated when:

$$\frac{V^2}{R} = 2V \,.\, u \sin f \tag{10}$$

is satisfied, where V is the average velocity in the river.

The intensity of the downward component of the current at the concave bank of a bend is measured by the difference:

$$\frac{v^2}{R} - \frac{v'^2}{R} \tag{11}$$

The greater this difference, the greater the downward component with high scouring power.

On the other hand considering the effect of the terrestrial rotation on a straight reach of the river one finds that the intensity of the downward component of the helicoidal current is measured by:

$$2\,v \,.\, u \sin f - 2\,v' \,.\, u \sin f \tag{12}$$

Introducing the average velocity V, (11) and (12) give:

$$\frac{v^2}{R} - \frac{v'^2}{R} = \frac{V^2}{R}\left[\left(\frac{v}{V}\right)^2 - \left(\frac{v'}{V}\right)^2\right]$$

and
$$2\,v \,.\, u \sin f - 2\,v' \,.\, u \sin f = 2\,V \,.\, u \sin f \left[\frac{v}{V} - \frac{v'}{V}\right]$$

If the assimilation defined by eq. (10) were valid for erosion phenomena, (11) and (12) would have the same value or

$$\frac{\frac{V^2}{R}\left[\left(\frac{v}{V}\right)^2 - \left(\frac{v'}{V}\right)^2\right]}{2\,V \,.\, u \sin f\left[\frac{v}{V} - \frac{v'}{V}\right]}$$

would be equal to 1. In fact this quantity results equal to $\frac{v}{V} + \frac{v'}{V}$ the value of which approximates rather 2 than 1.

As a result the considered assimilation cannot be quantitatively valid in so far as erosion phenomena are concerned, for the scouring power of the effect of terrestrial rotation is increased hereby.

In reduced scale tidal models a notable reason of disparity between model and prototype may be the Coriolis acceleration. In the model the influence of this acceleration is negligible while in the prototype it may be considerable.

Different solutions have been proposed in order to remove this difficulty.

A nice but rather expensive and technically difficult solution seems to be given by making the model rotate. Some preliminary investigations with a turning model have been carried out in France.

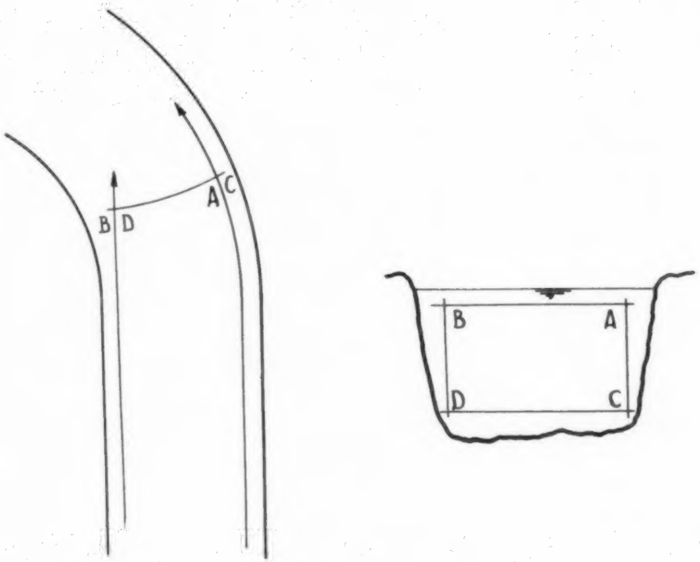

Fig. 1.

It has also been proposed for distorted models of tidal rivers to correct the model with regard to the Coriolis force by increasing the curvatures in plan according to formula (10).

If the model is one with movable bed it should be realised that in this manner the erosion due to Coriolis acceleration will be exaggeratedly increased.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246 (HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1263(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303 (ST4), 1304(ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318 (ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338(SA4), 1339(CO1) 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

c. Discussion of several papers, grouped by Divisions.